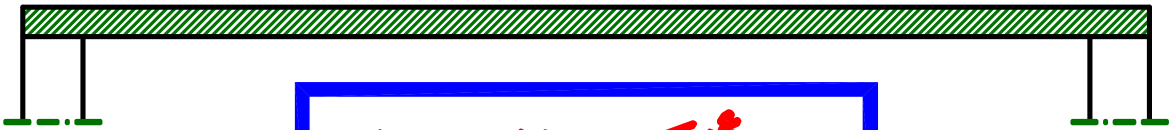
Flat Slabs البلاطات المسطحة



نسألكم الدعاء

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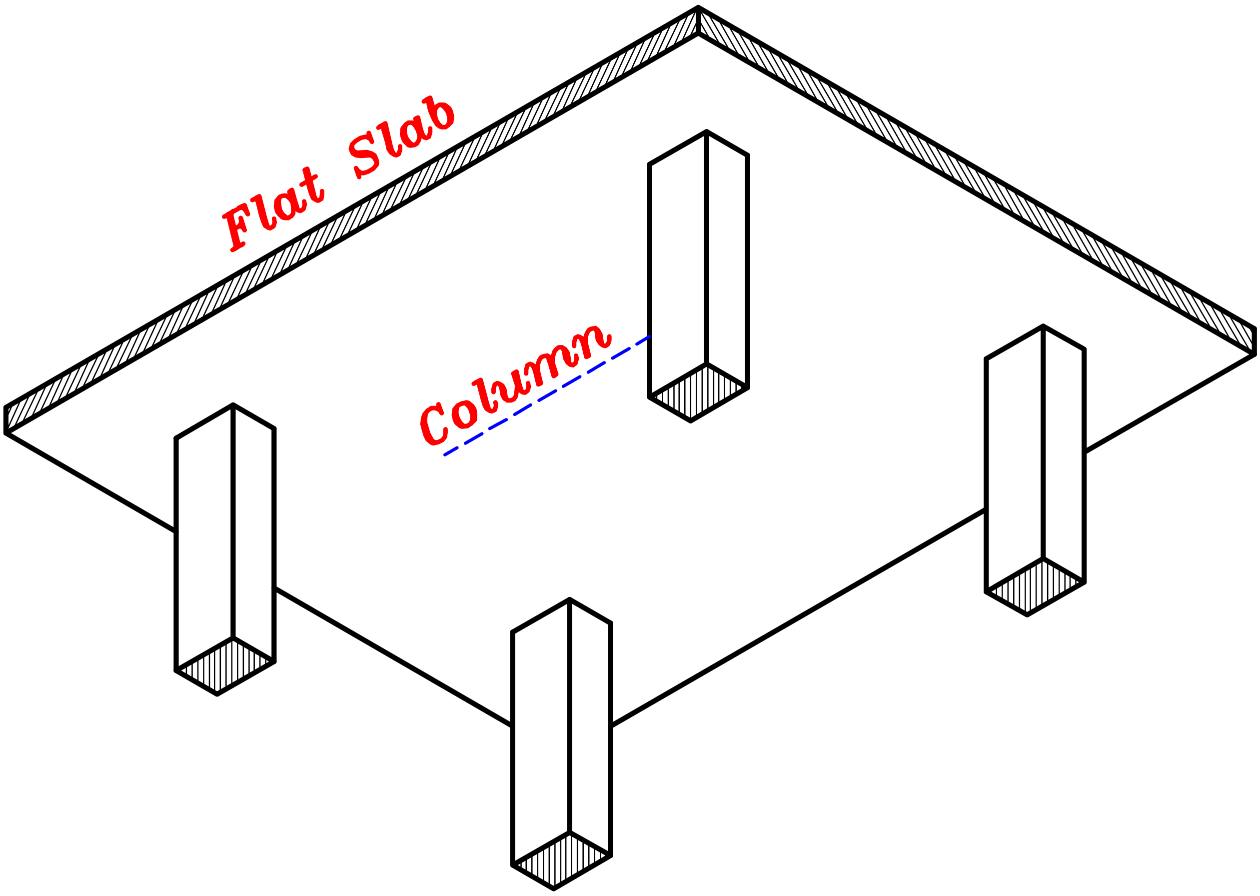
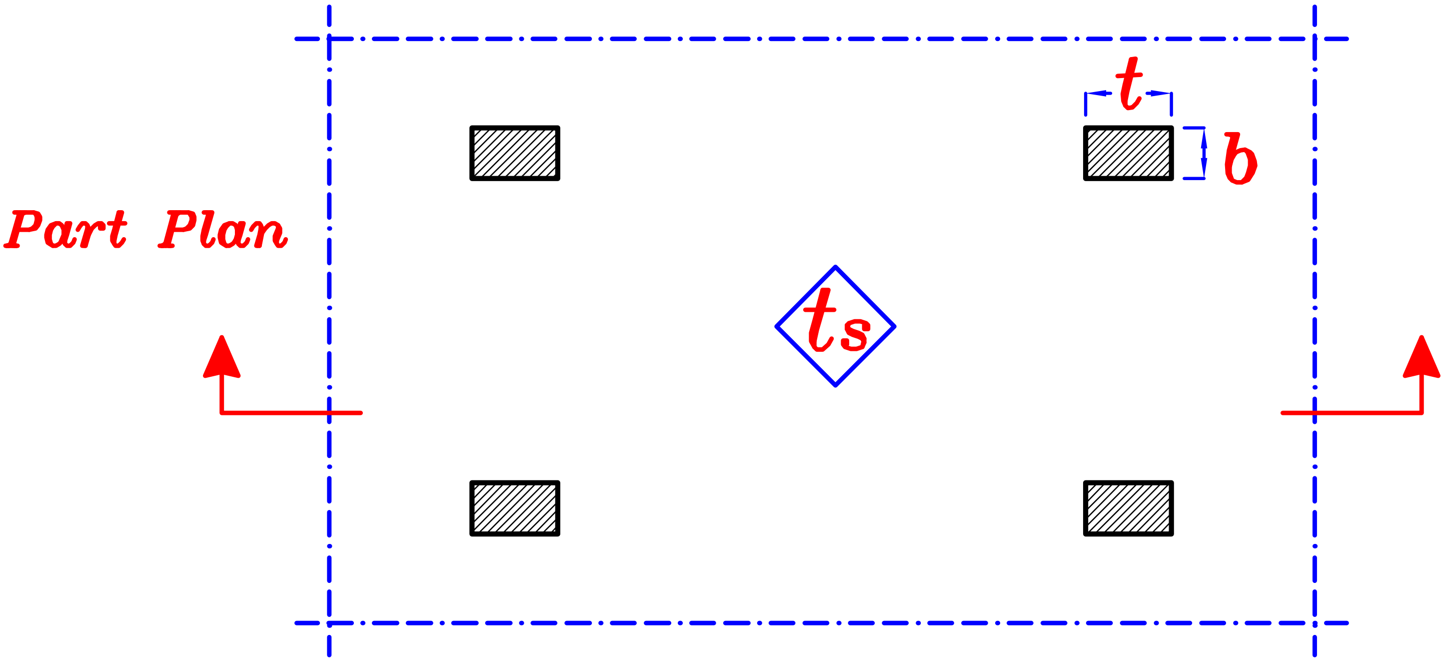
اذا حملت تطبيق **RC Structures** YASSER EL LEATHY على تليفونك المحمول او اللوح السطحى ستستطيع أن تشغل أفلام شرح للمقاطع التى تحتوى على رمز

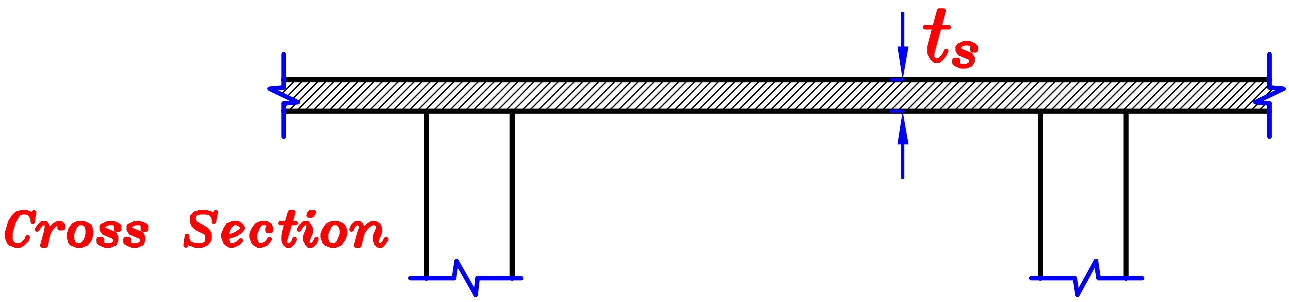
Flat Slab. Table of Contents.

Introduction.

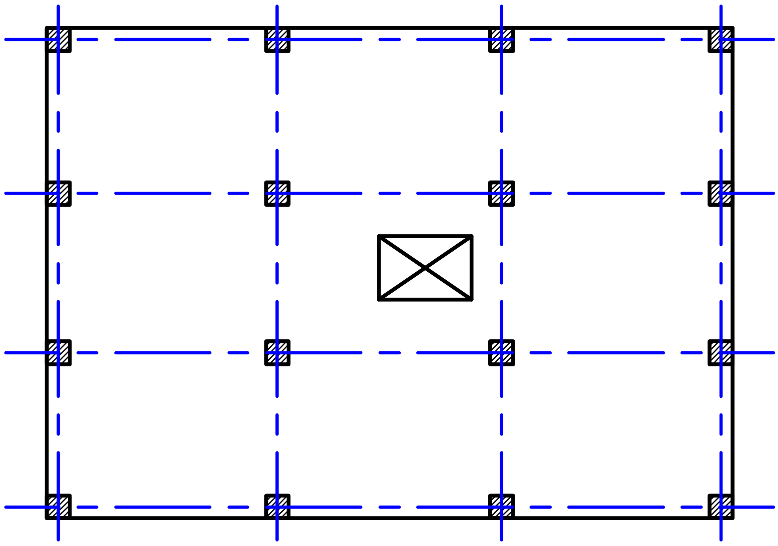
ال (**Flat Slabs**) هى عباره عن بلاطات مسطحه ترتكز على الاعمده مباشره . (أى لا توجد بها كمرات)

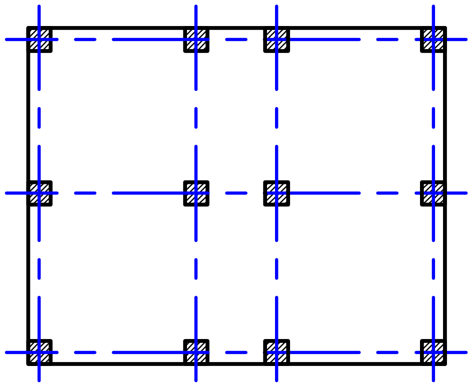


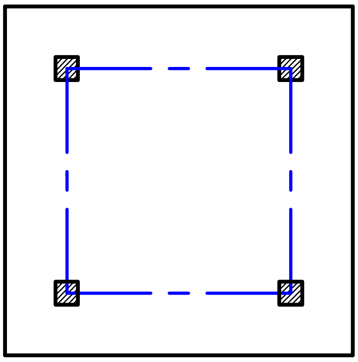


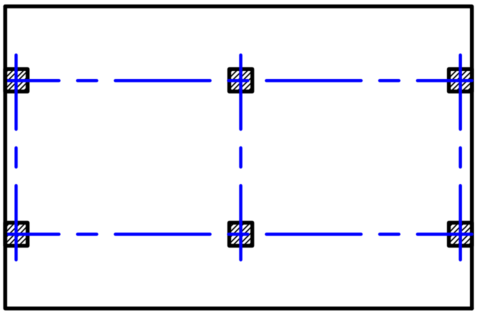


فى هذا الملف سيتم دراسه البلاطات ال **Flat Slab** ذات الاشكال المنتظمه .
اى التى تقع فيها الاعمده على محاور مستقيمه موازيه و عموديه على بعض .

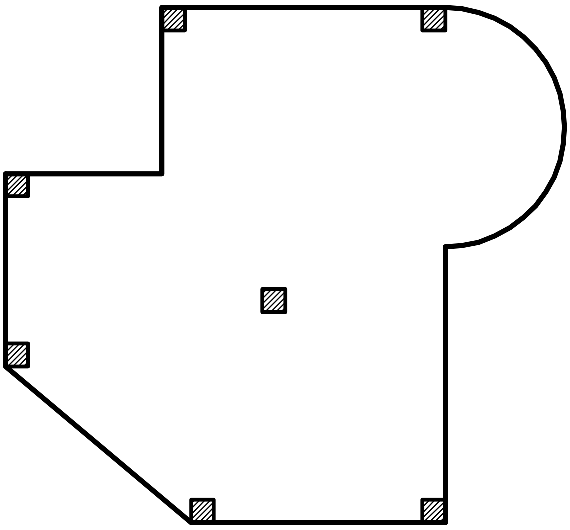


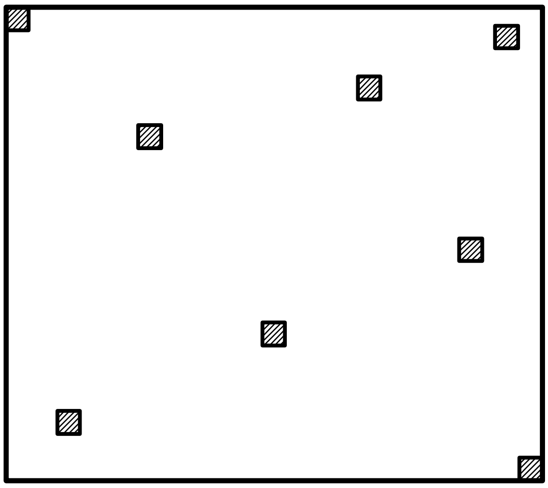




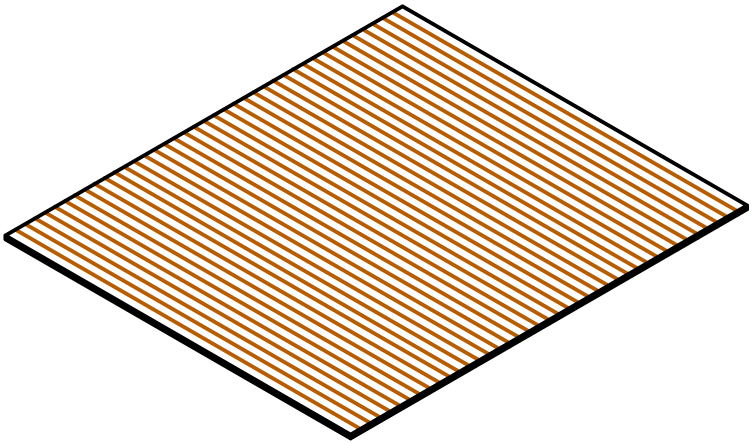


اما البلاطات ال **Flat Slab** ذات الاشكال غير المنتظمه
او محاورها ليست موازيه و عموديه على بعض
فسيتم تحليلها على الكمبيوتر و لن يتم دراستها فى هذا الملف .

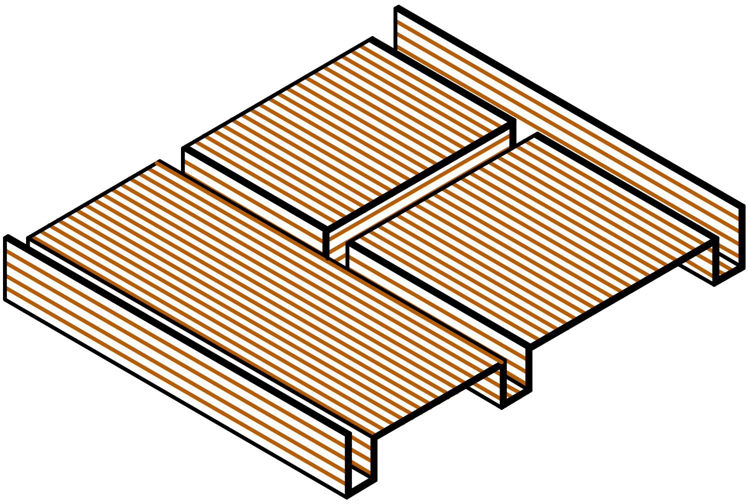




تتميز ال (**Flat Slabs**) بسهوله التنفيذ لان الشده الخشبيه للبلاطه افقيه و مستويه ای لا یوجد بها سقوط للكمرات .

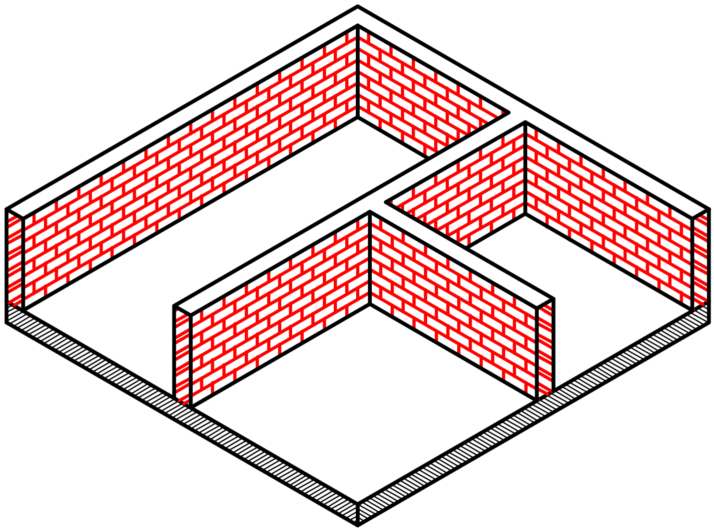


شده خشبيه لبلاطه **Flat**

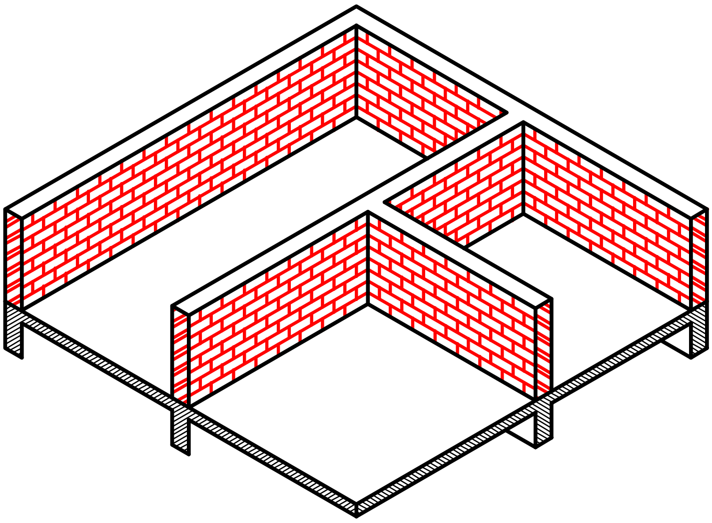


شده خشبيه لبلاطه **Solid**

و تتميز ال (**Flat Slabs**) ايضا انها تُصمم على وضع الحوائط فى اى مكان على البلاطه أى من الممكن تغيير تقسيم الحوائط الداخليه للمبنى دون اى اعتبارات انشائيه .



فى ال **Flat Slab** ممكن وضع الحائط اى مكان فوق البلاطه



فى ال **Solid Slab** يجب وضع الحائط فوق الكمره مباشره

Concept of load transfer.

ينتقل الحمل من البلاطه الـ **Flat Slab** الى الاعمده عن طريق :

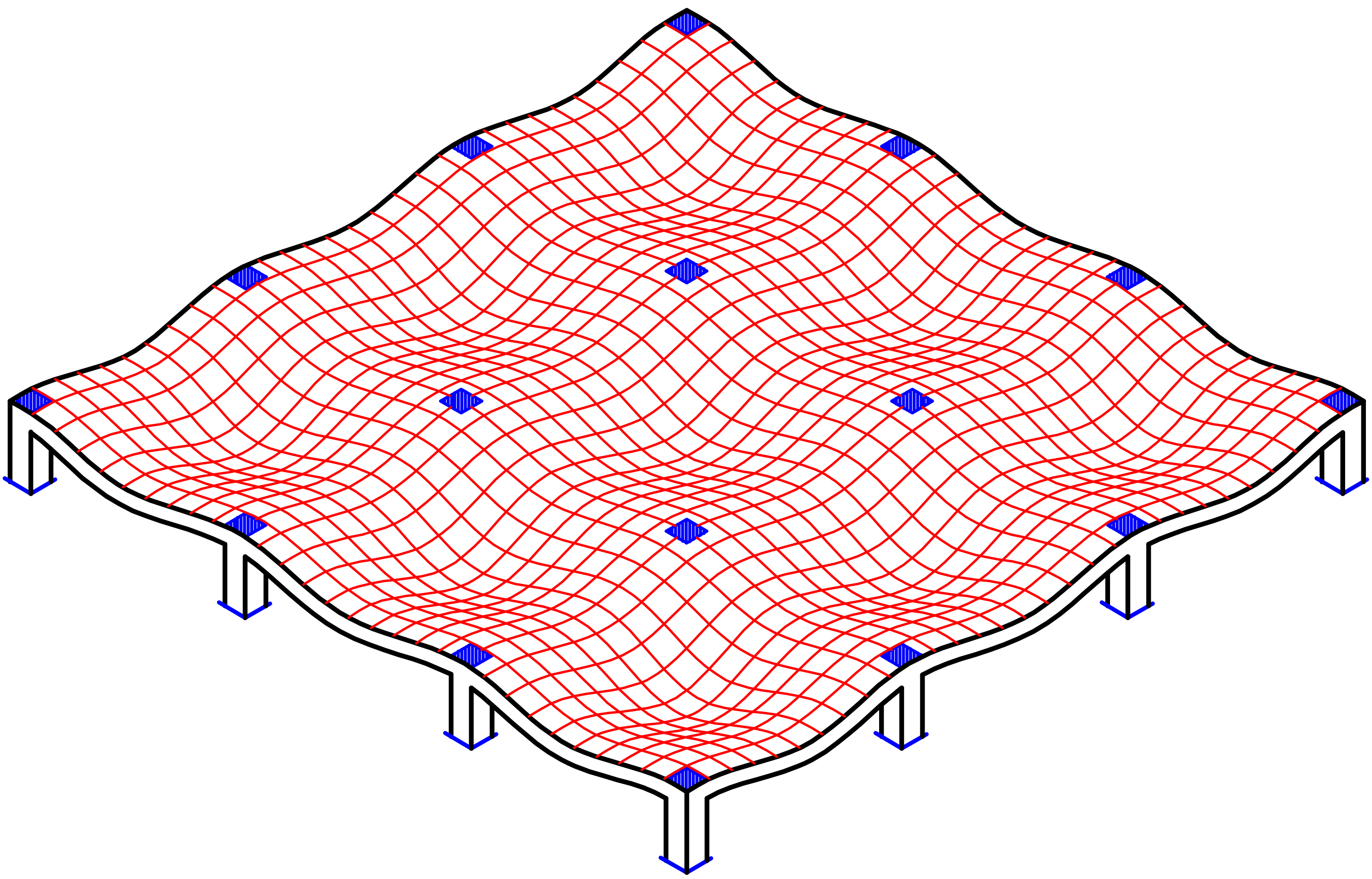
١ـ حدوث انحناء فى البلاطه **(Bending Moment)**

٢ـ حدوث قوى قص **(Punching Shear)** تنقل الاحمال من البلاطه الى العمود مباشره .

اذاً يجب تصميم البلاطه بحيث تتحمل الانحناء الواقع عليها و ان يكون الـ **Deflection** الواقع عليها فى حدود المسموح به .

و يجب ان تتحمل البلاطه قوى القص و القص الثاقب **(Punching Shear)** الواقع عليها .

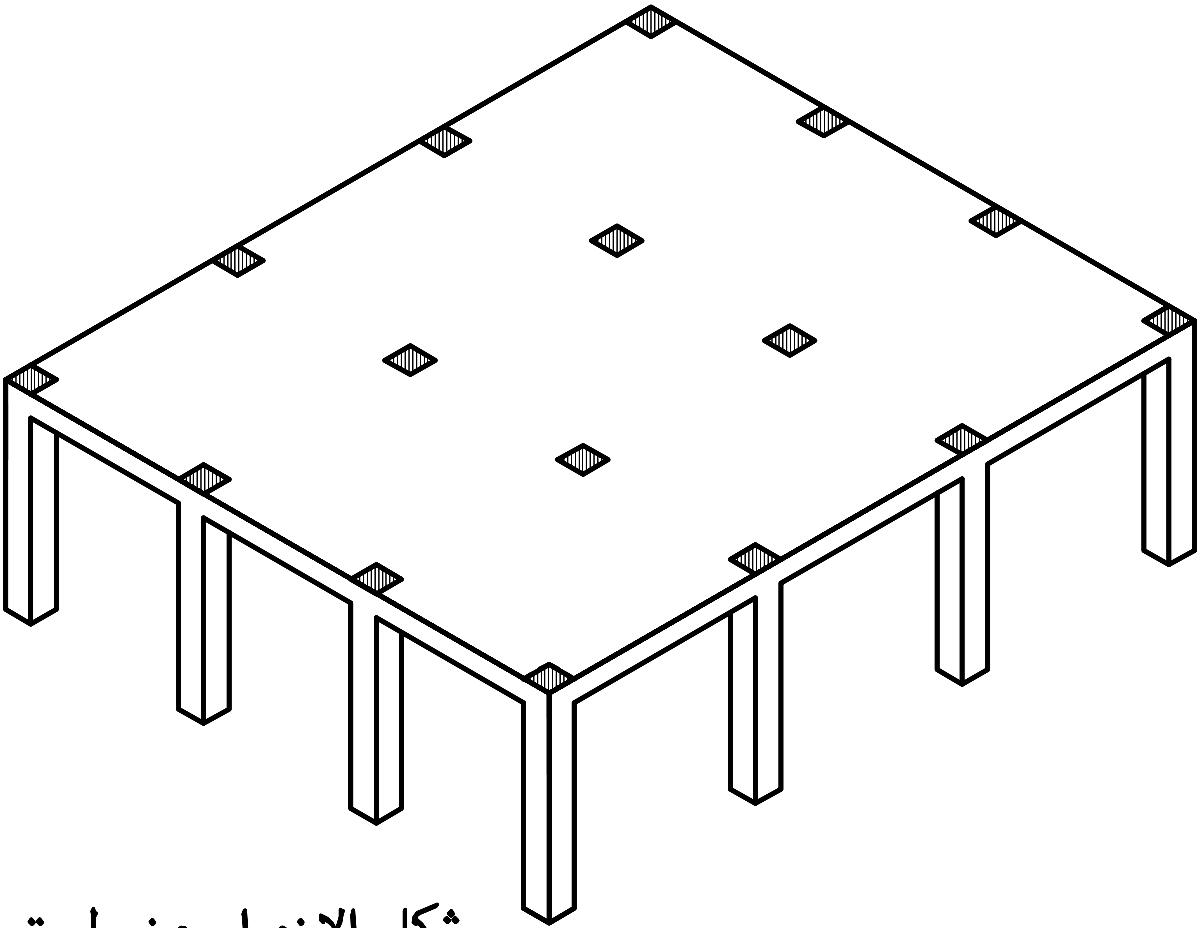
و بما انه يحدث انحناء للبلاطه فى الاتجاهين اذاً يوجد عزم على البلاطه فى الاتجاهين .



شكل الـ **Deflection** الواقع على البلاطه نتيجه الاحمال الرأسيه

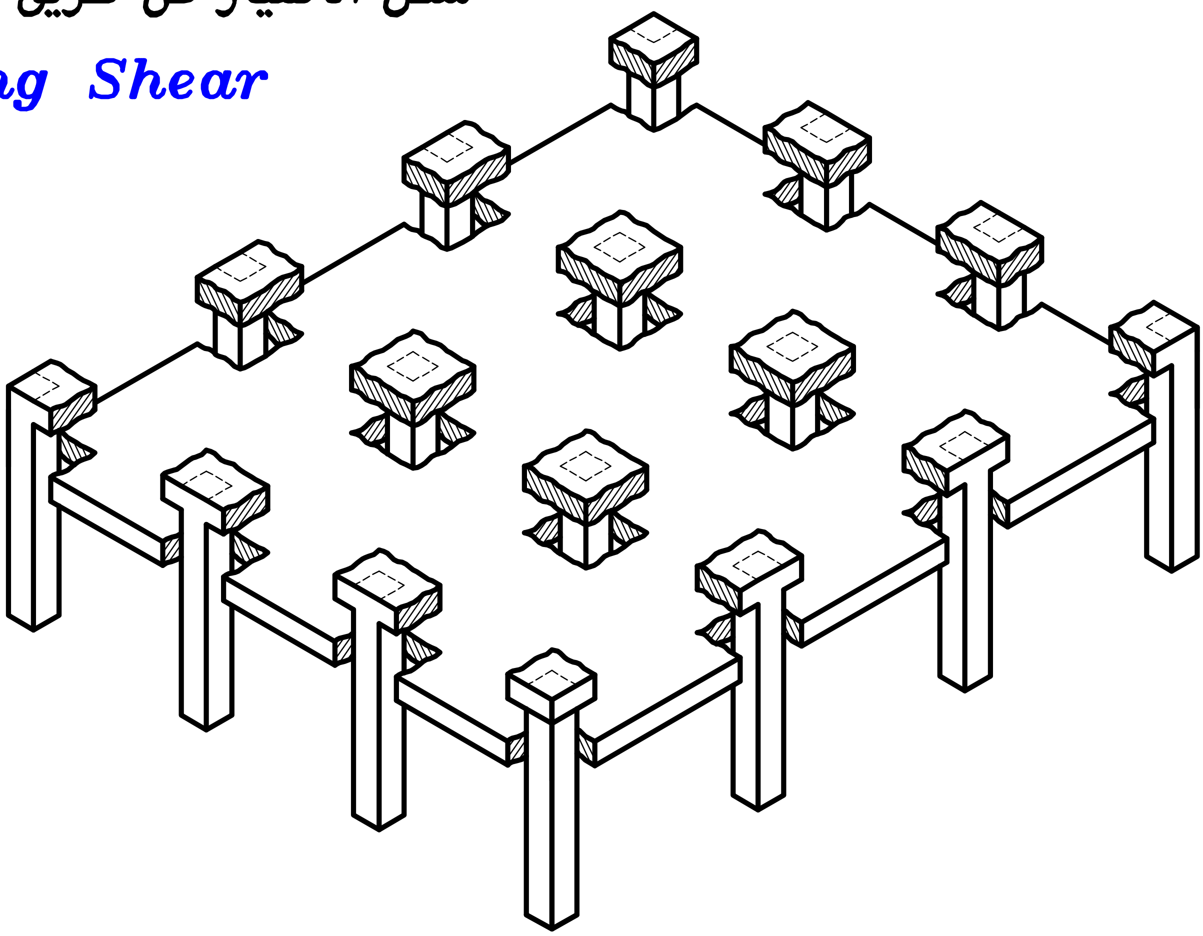
ينتقل الحمل من البلاطه الى العمود عن طريق القص الثاقب **Punching Shear**

لذا يجب ضمان ان البلاطه ستتحمل ال **Punching**

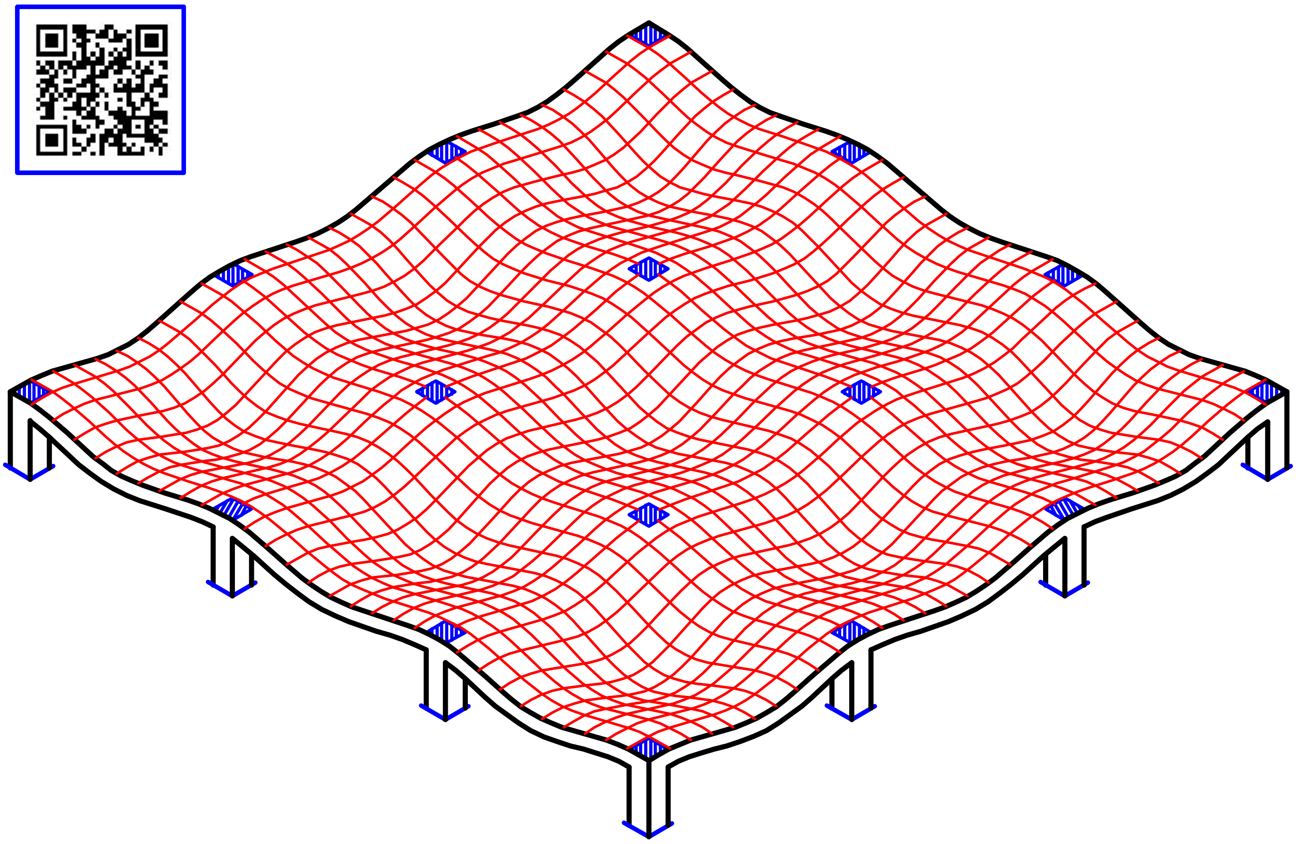


شكل الانهيار عن طريق **القص الثاقب**

Punching Shear

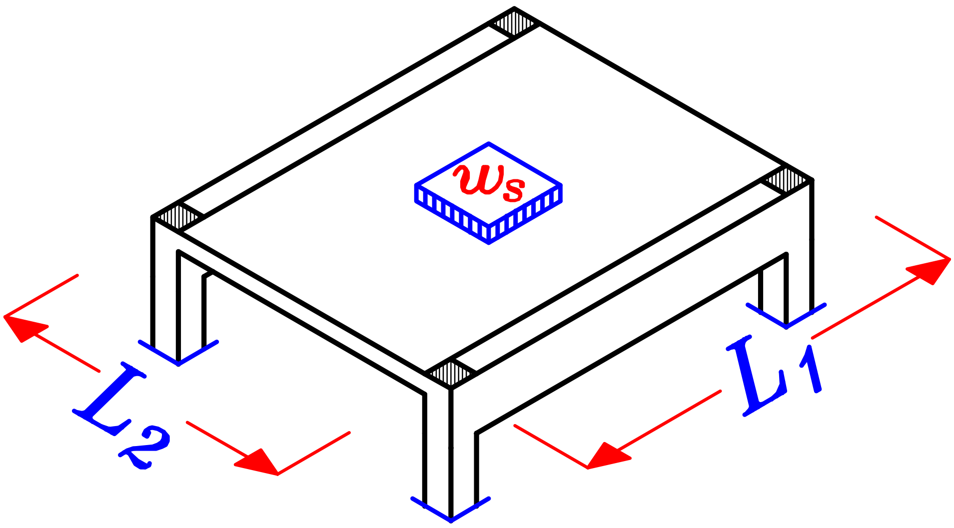


الانحناء فى ال **Flat Slab** يحدث فى الاتجاهين كأن كل اتجاه منهم عباره بلاطه **One way solid** منفصله عن الاخرى .



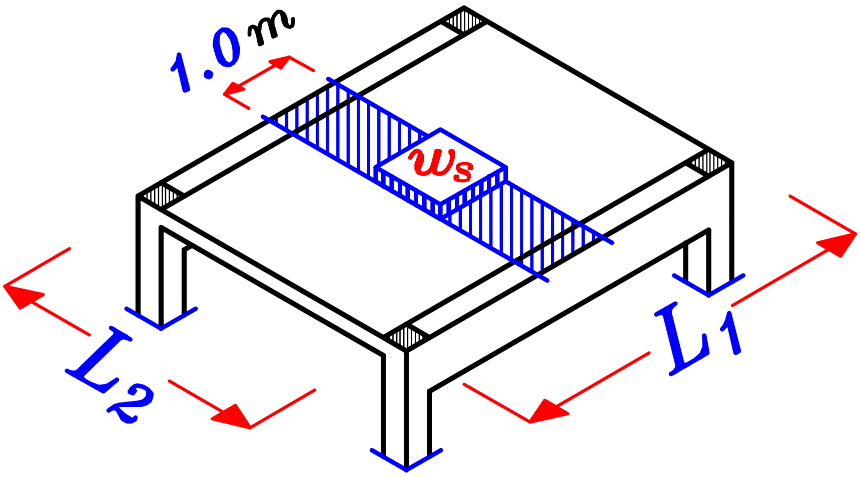
فى البلاطات ال **One way solid** لكى يذهب الحمل الى العمود يحدث انحناء فى الاتجاهين اتجاه فى البلاطه و الاتجاه العمودى عليه للكمره .

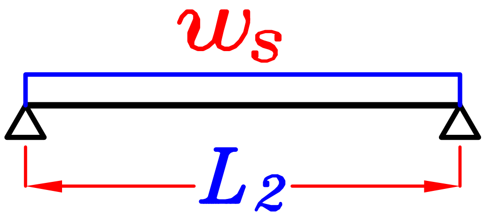
وزن المتر المربع على البلاطه يساوى w_s



بأخذ شريحه فى البلاطه

فى اتجاه L_2 عرضها $1.0\ m$

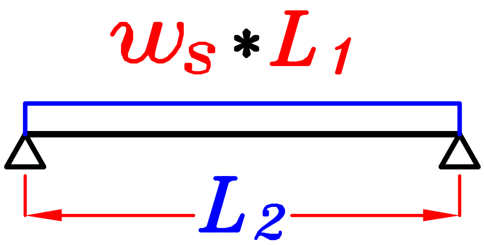


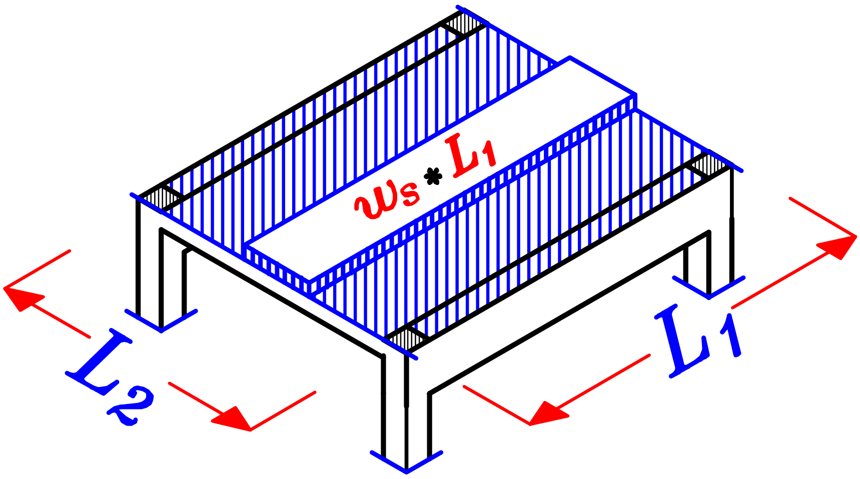


$$M = \frac{w_s * L_2^2}{8}$$ ---------- ١,٠ م – عزم شريحه بلاطه عرضها

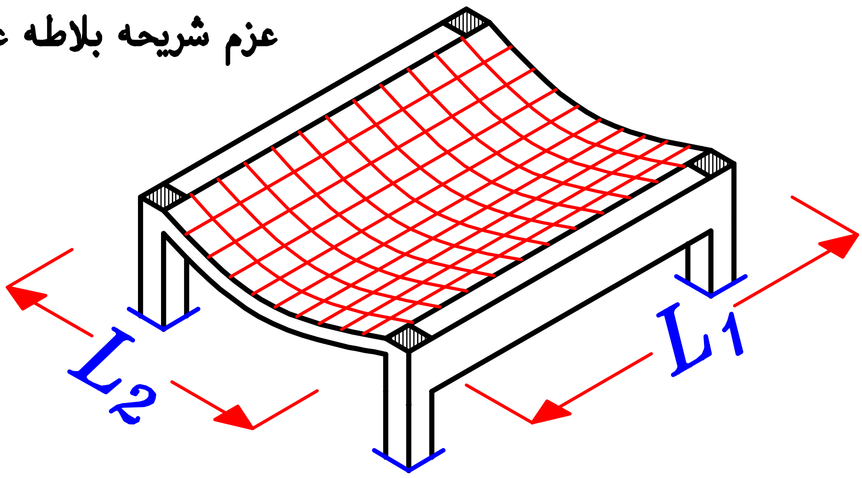
لكن اذا اخذنا شريحه بعرض البلاطه كلها L_1

سيكون الحمل فى المتر الطولى على هذه الشريحه يساوى $w_s * L_1$





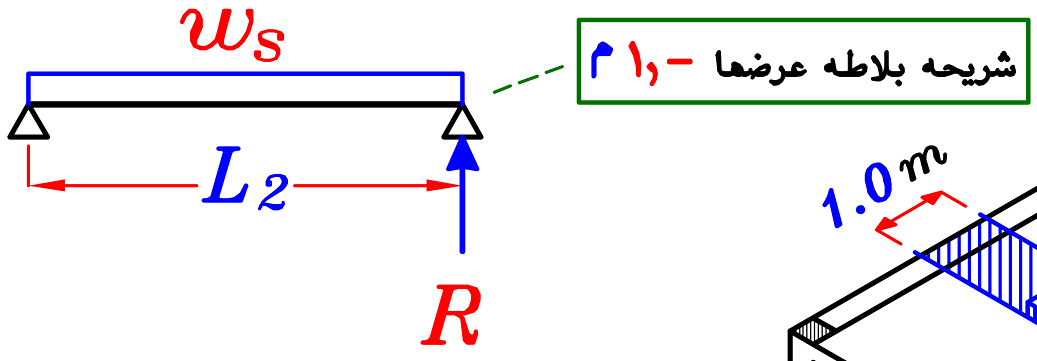
M_1 ------ عزم شريحه بلاطه عرضها L_1



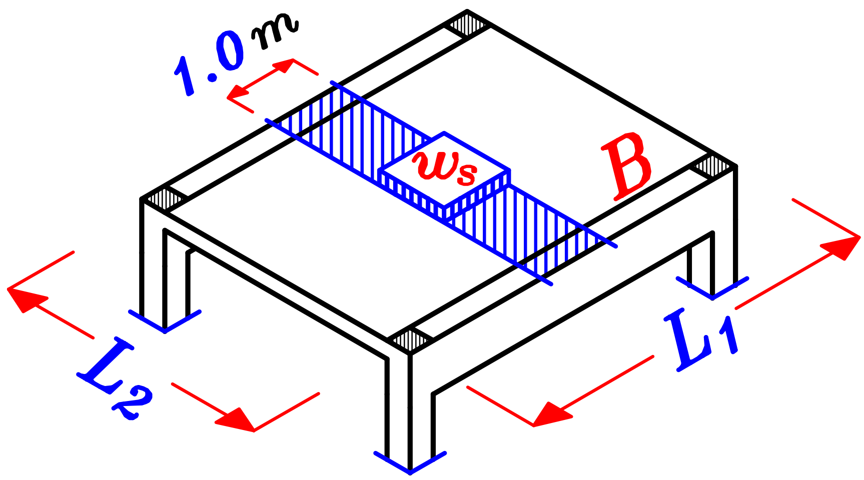
$$\boxed{M_1 = \frac{(w_s * L_1) * L_2^2}{8}}$$

لحساب حمل البلاطه على متر طولى من الكمره B

نأخذ $Reaction$ شريحه بلاطه عرضها ١,٠٠ م



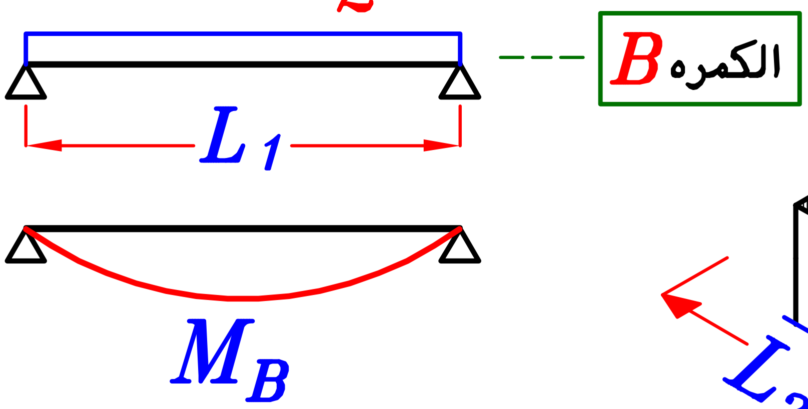
$$R = \frac{w_s * L_2}{2}$$

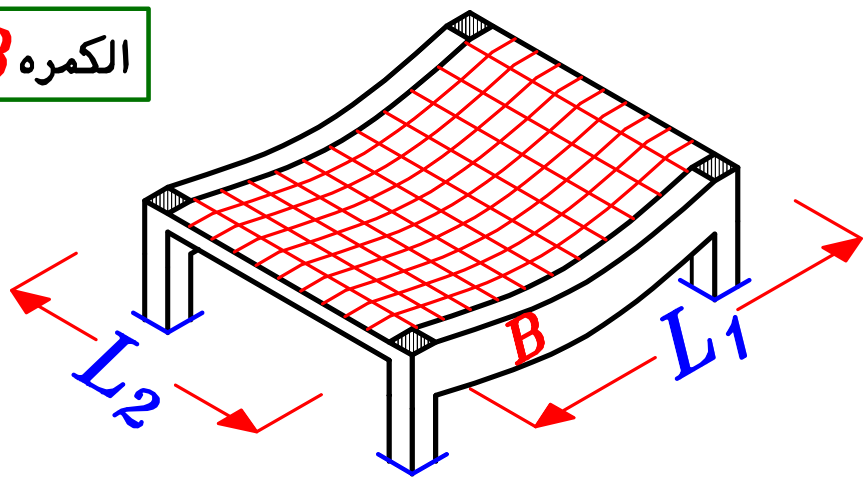


بوضع الحمل المنتظم R على الكمره B

و حساب العزم M_B على الكمره B

$$w = R = \frac{w_s * L_2}{2}$$

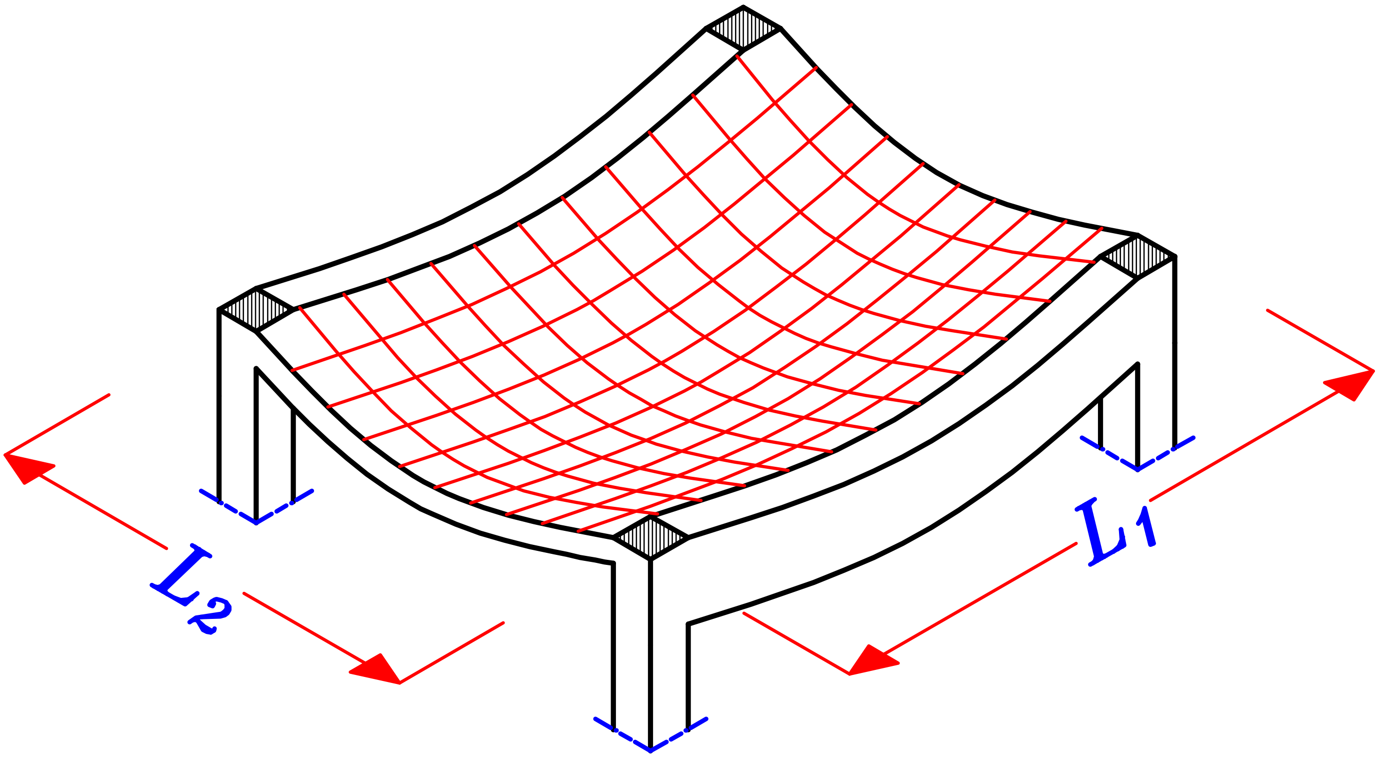




$$M_B = \frac{\left(\frac{w_s * L_2}{2}\right) * L_1^2}{8} = \frac{(w_s * L_2) * L_1^2}{16}$$

اذاً مجموع العزم على الكمرتين معا يساوى M_2

$$M_2 = \frac{(w_s * L_2) * L_1^2}{8}$$



اذاً لكى يذهب حمل البلاطه الـ One way solid الى الاعمده يجب ان يحدث انحناء (moment) فى الاتجاهين .

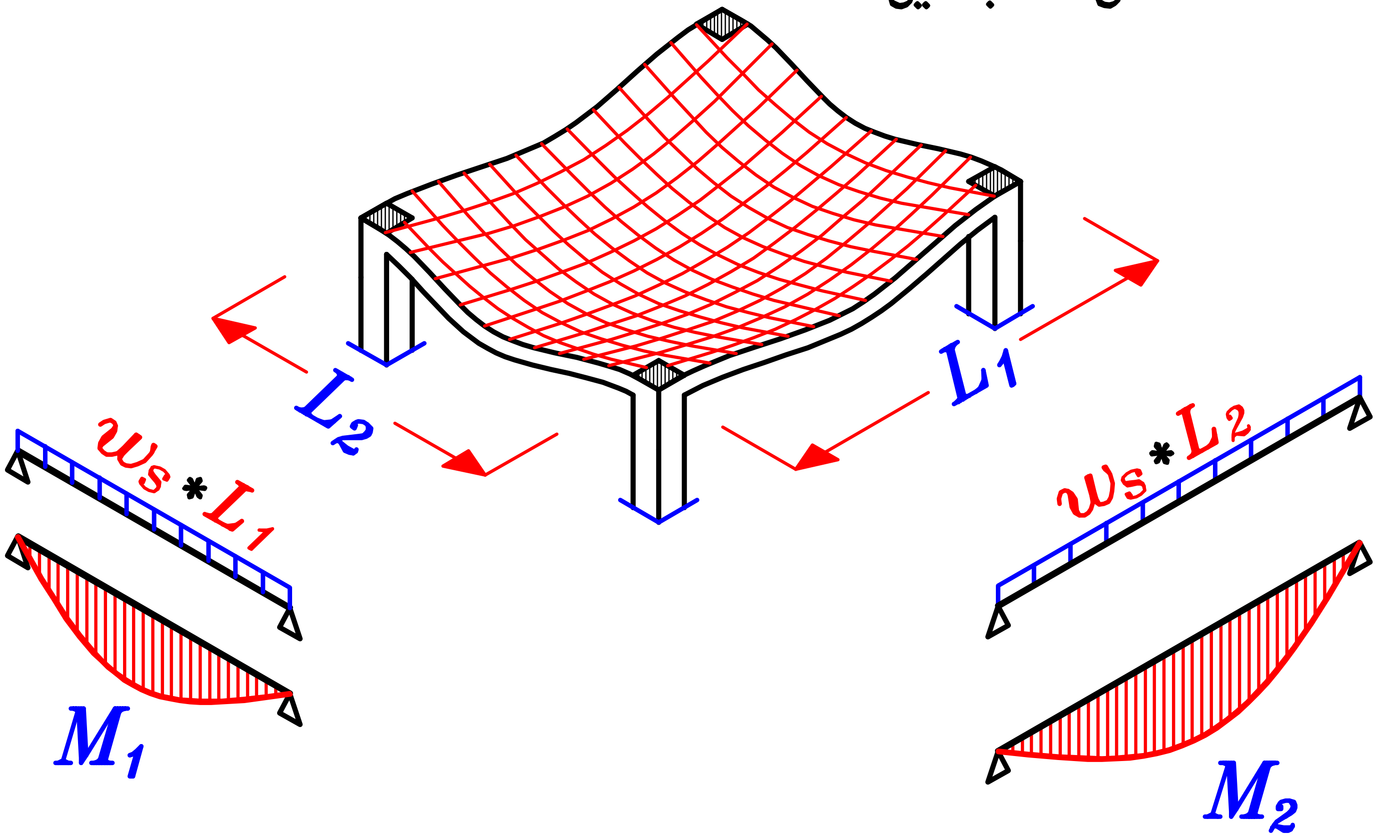
$$M_1 = \frac{(w_s * L_1) * L_2^2}{8}$$

عزم على البلاطه كلها
فى اتجاه L_2

$$M_2 = \frac{(w_s * L_2) * L_1^2}{8}$$

عزم على الكمرتين معا
فى اتجاه L_1

ايضاً فى البلاطات ال Flat Slab لكى يذهب حمل البلاطه الى العمود يحدث انحناء فى الاتجاهين



$$M_1 = \frac{(w_s * L_1) * L_2^2}{8}$$

عزم على البلاطه كلها
فى اتجاه L_2

$$M_2 = \frac{(w_s * L_2) * L_1^2}{8}$$

عزم على البلاطه كلها
فى اتجاه L_1

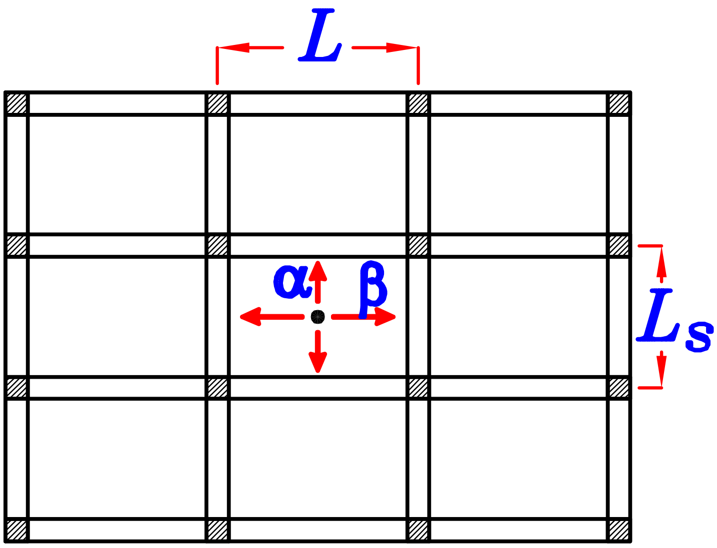
اذا فى ال Flat Slab وزن البلاطه w_s بالكامل (بدون توزيع للحمل مثل ال Solid)
يسبب moment على البلاطه فى الاتجاه القصير .
و مره اخرى وزن البلاطه w_s بالكامل يسبب moment على البلاطه فى الاتجاه الطويل .
أى لا يوجد α , β فى البلاطات ال Flat Slab

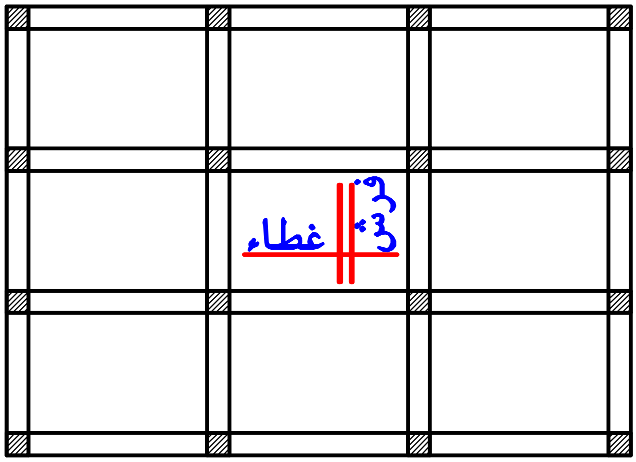
فى البلاطات ال **Solid** تكون البلاطه محموله على كمرات و الكمرات محموله على اعمده لذا ينتقل الحمل من البلاطه الى الكمرات اولا ثم الى الاعمده .

لذا النسبه الاكبر من الحمل α غالبا تتوزع فى الاتجاه الاقصر لان ال **Stiffness** له اكبر .

و النسبه الاقل من الحمل β تتوزع فى الاتجاه الاطول .

و عاده يكون تسليح الاتجاه القصير هو **الفرش** و الاتجاه الطويل هو **الغطاء**





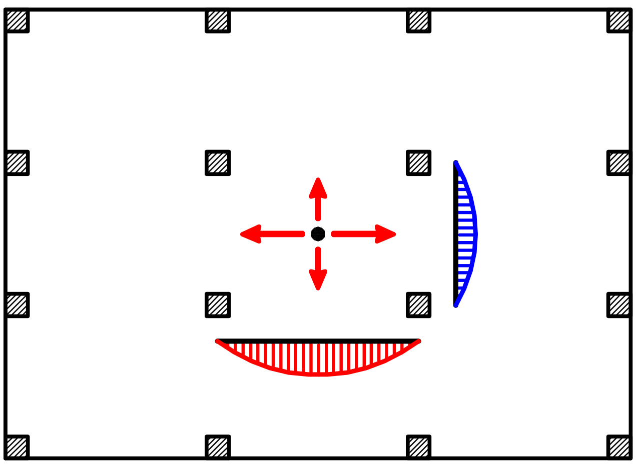
أما فى البلاطه ال **Flat Slab**

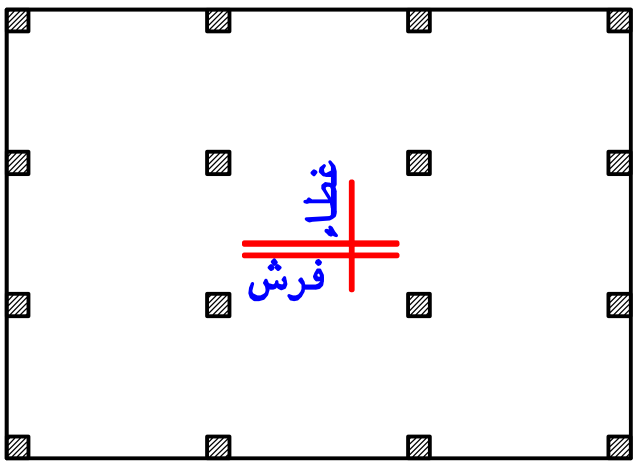
الحمل بالكامل يسبب **moment** فى الاتجاه القصير كأنها بلاطه **one way** فى الاتجاه القصير

و نفس الحمل بالكامل يسبب **moment** فى الاتجاه الطويل كأنها بلاطه **one way** فى الاتجاه الطويل

و بالتالى يكون ال **moment** فى الاتجاه الطويل اكبر بكثير

و يكون التسليح فى الاتجاه الطويل هو ال **الفرش** و الاتجاه القصير هو **الغطاء**





مقارنه بين ال Solid Slab و ال Flat Slab

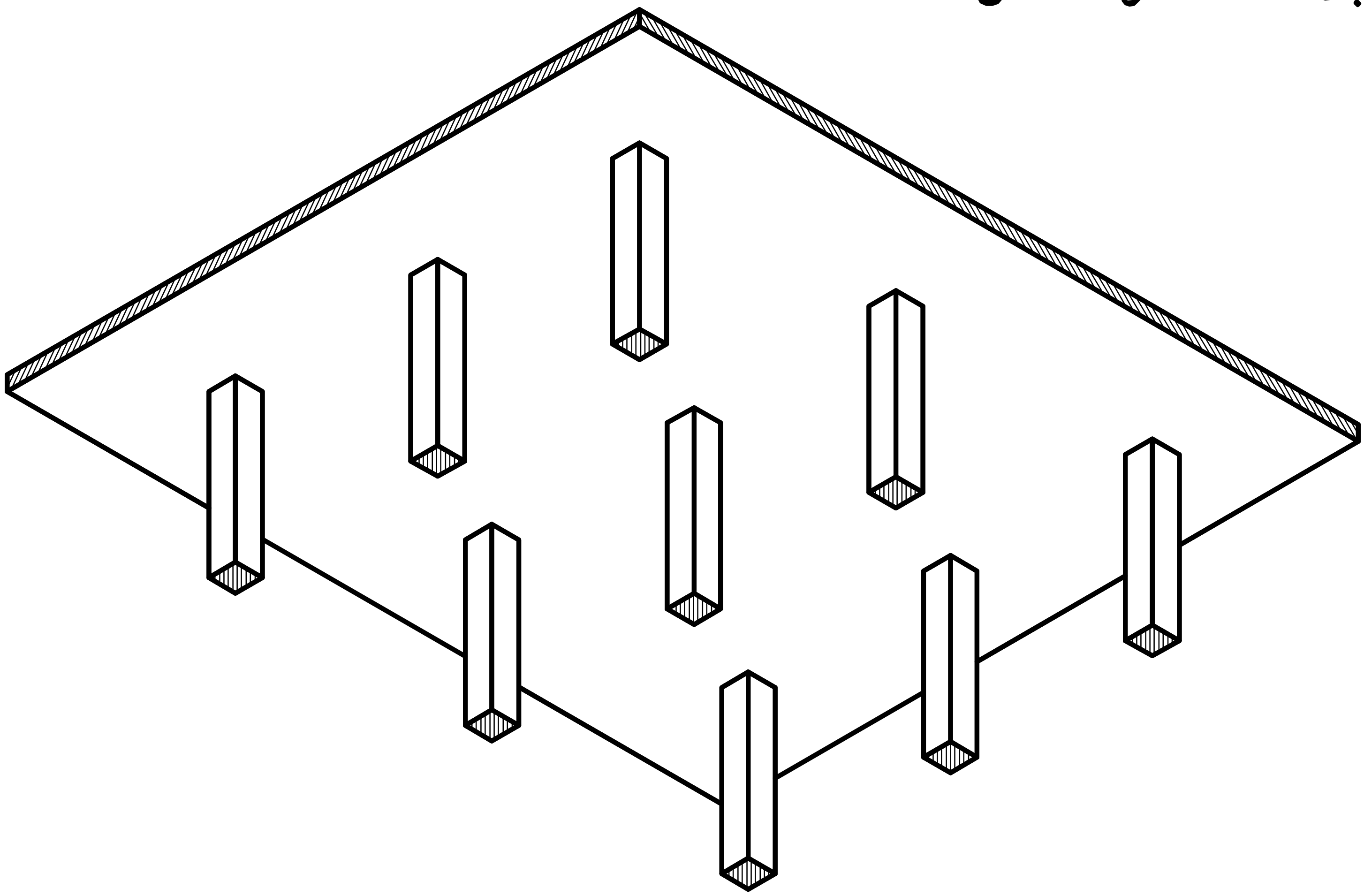
Flat slabs	Solid slabs
– هى بلاطات محموله مباشره على الاعمده .	– هى بلاطات محموله على كمرات و الكمرات محموله على اعمده .
– معماريا افضل لعدم وجود كمرات .	– توجد بها كمرات .
– تفضل فى حاله البحور الكبيره . (*up to* **10.0** *m span.*)	– تفضل فى البحور الصغيره . (*up to* **4.50** *m For* L_s)
– تفضل فى حاله الاحمال الكبيره . مثل الجراجات .	– تفضل فى الاحمال العاديه .
– الشده الخشبيه أسهل فى التنفيذ . لانها أفقيه و لا يوجد بها سقوط كمرات	– الشده الخشبيه أصعب فى التنفيذ . نتيجه لوجود سقوط للكمرات .
– تخانه البلاطه t_s كبيره نسبياً لمقاومه (−Ve) *moment* & *punching shear*	– تخانه البلاطه t_s صغيره نسبياً لان *moment* (−Ve) أقل من حاله *Flat Slab* و لعدم وجود *punching shear*
– كميه حديد التسليح كبيره نسبياً بالمقارنه بال *Solid slab*	– كميه حديد التسليح صغيره نسبياً بالمقارنه بال *Flat slab*
– الاتجاه الطويل هو الاتجاه الذى يكون فيه عزوم *moment* أكبر . فيوضع فيه كميه الحديد الاكبر . و يكون التسليح فى هذا الاتجاه هو الفرش .	– غالبا يكون الاتجاه القصير هو الاتجاه الذى يكون فيه عزوم *moment* أكبر . فيوضع فيه كميه الحديد الاكبر . و يكون التسليح فى هذا الاتجاه هو الفرش .

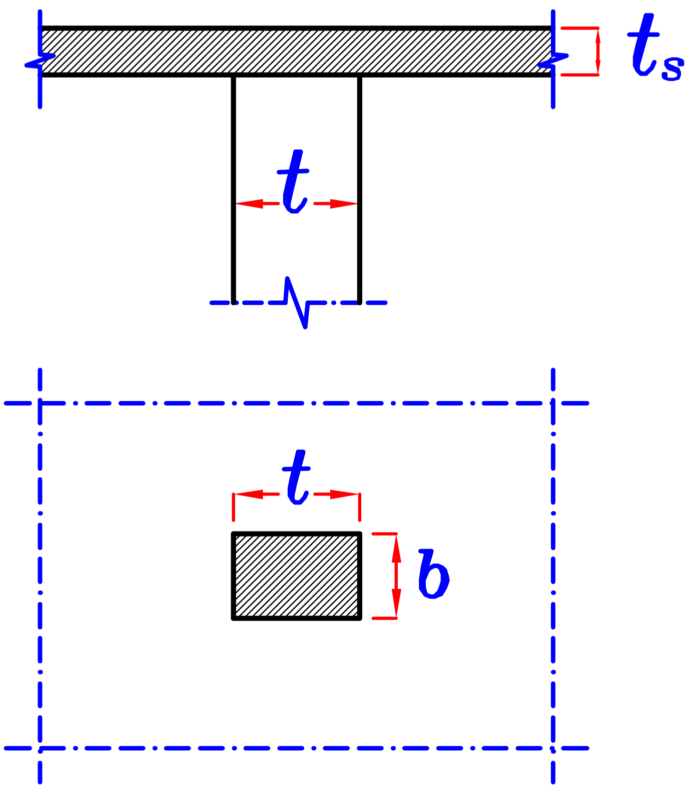
Types of Flat Slabs.

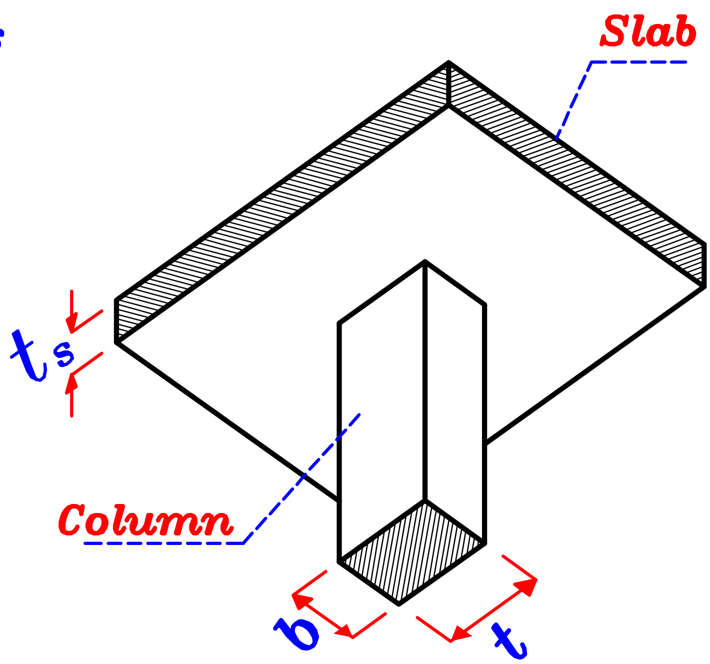
1 – Ordinary Flat Slab.

(Flat Slab without Drop Panel or Column Head.)

بلاطه محموله على اعمده فقط.





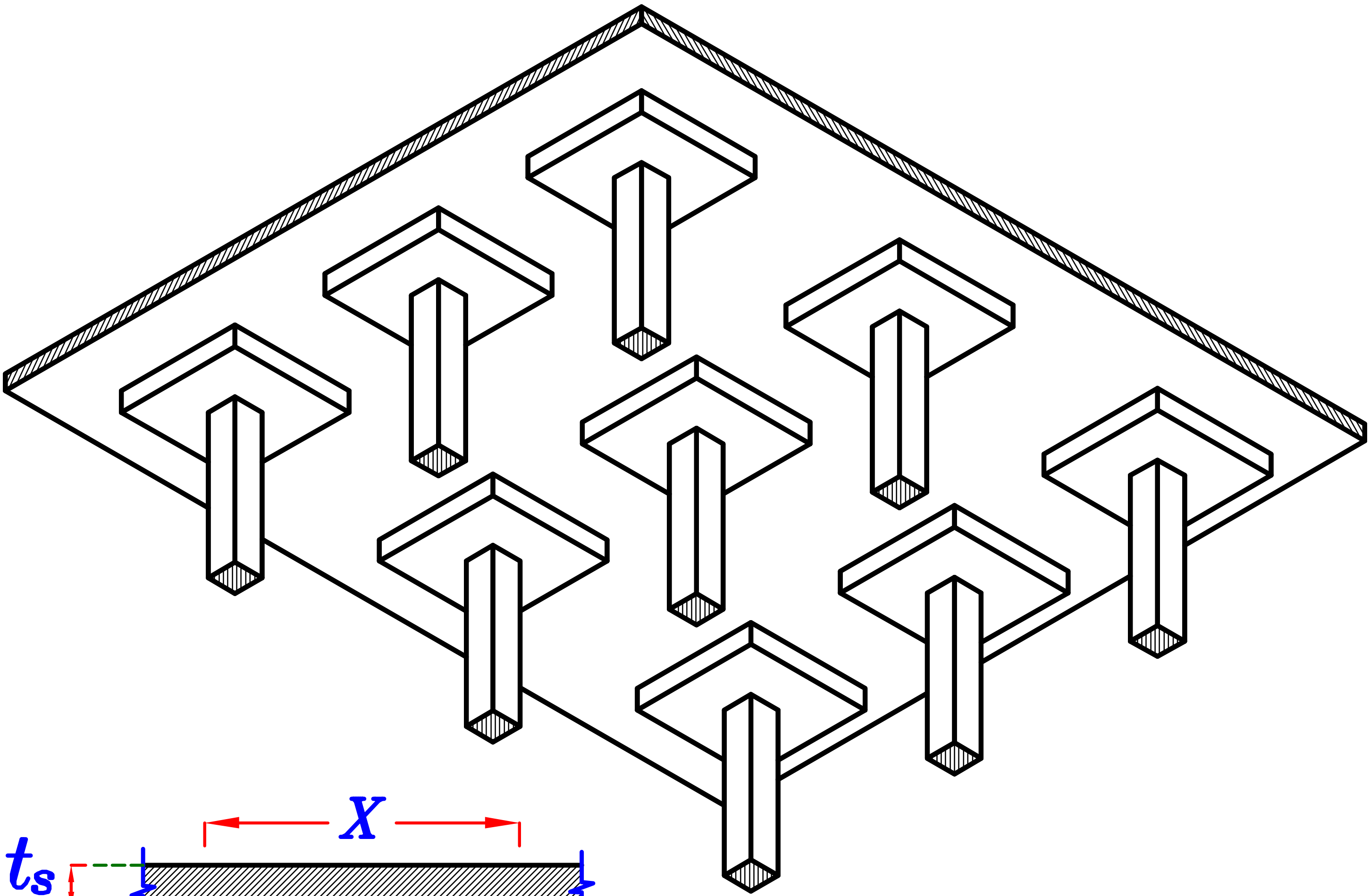


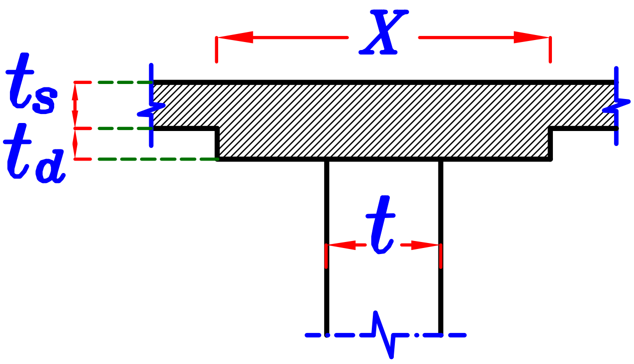
2-Flat Slab with Drop Panel.

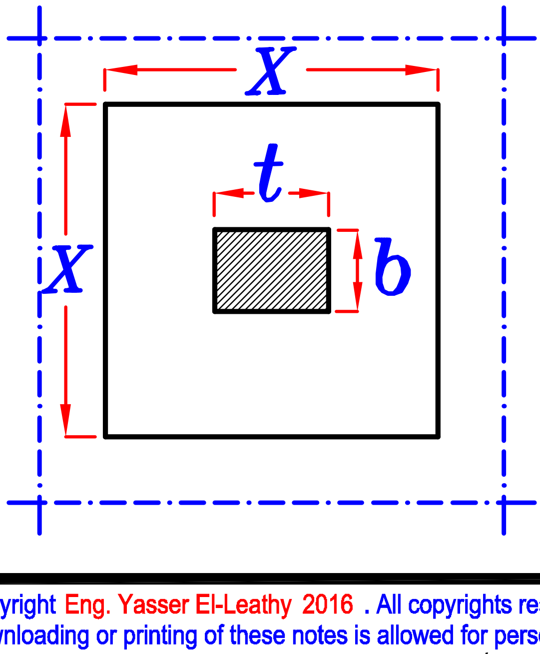
(**Used For Large Spans to resist high (-Ve) moments**)

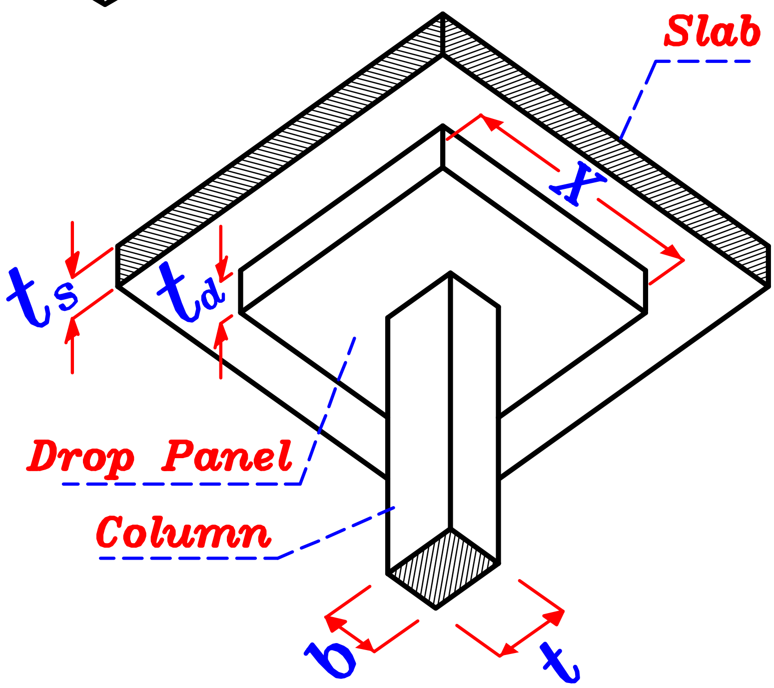
فى البحور الكبيره يزيد الـ **moments (-Ve)** جدا لذا بدلا من زياده تخانه البلاطه كلها

يفضل زياده تخانه البلاطه فوق الاعمده فقط للتوفير . **لكن شدتها الخشبيه صعبه فى التنفيذ .**





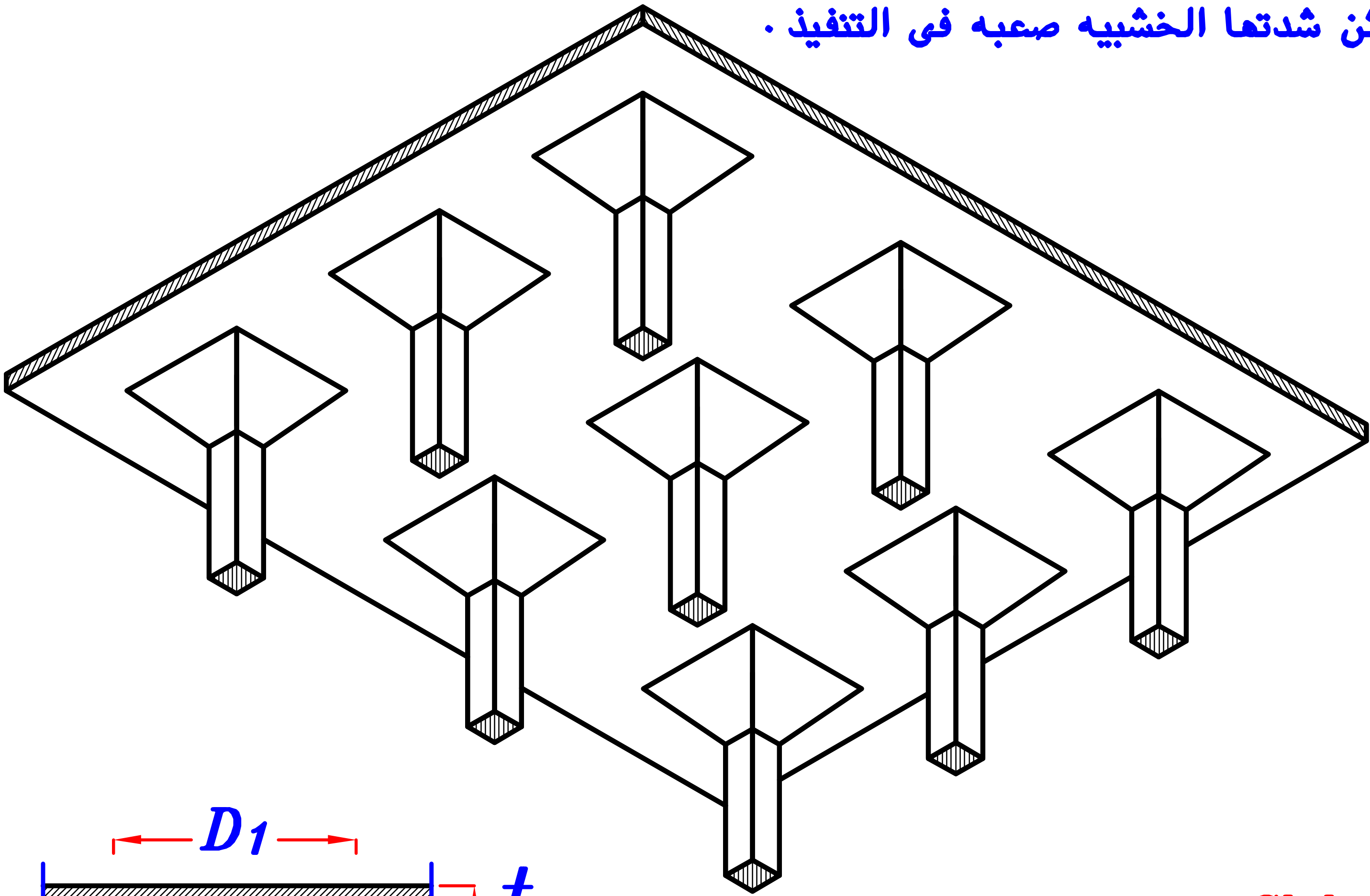


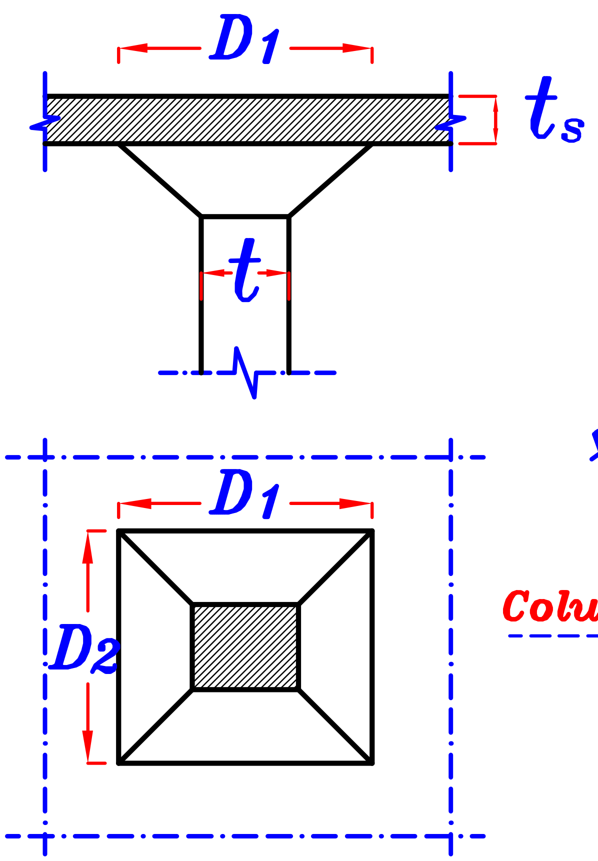


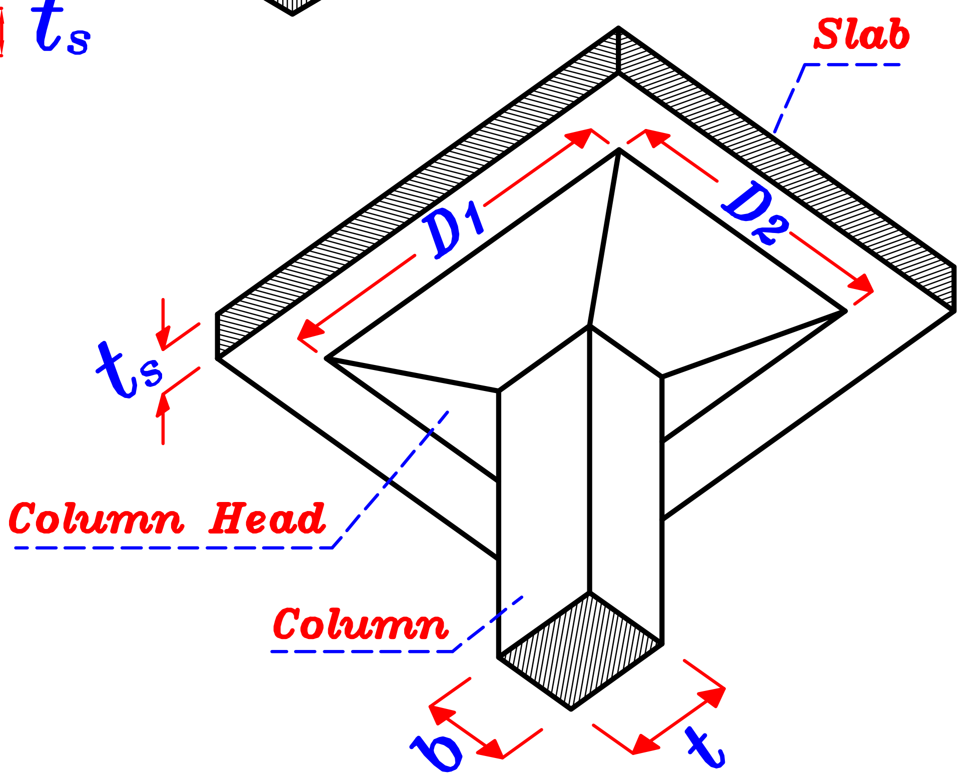
3-Flat Slab with Column Head.

(Used For Heavy Loads to avoid punching)

فى وجود احمال عاليه على البلاطه من الممكن حدوث ثقب **punching** للبلاطه عند منطقه الاعمده لذا بدلا من زياده تخانه البلاطه كلها يفضل عمل تاج للعمود **Column Head** للتوفير لكن شدتها الخشبيه صعبه فى التنفيذ .







4-Flat Slab with Drop Panel & Column Head.

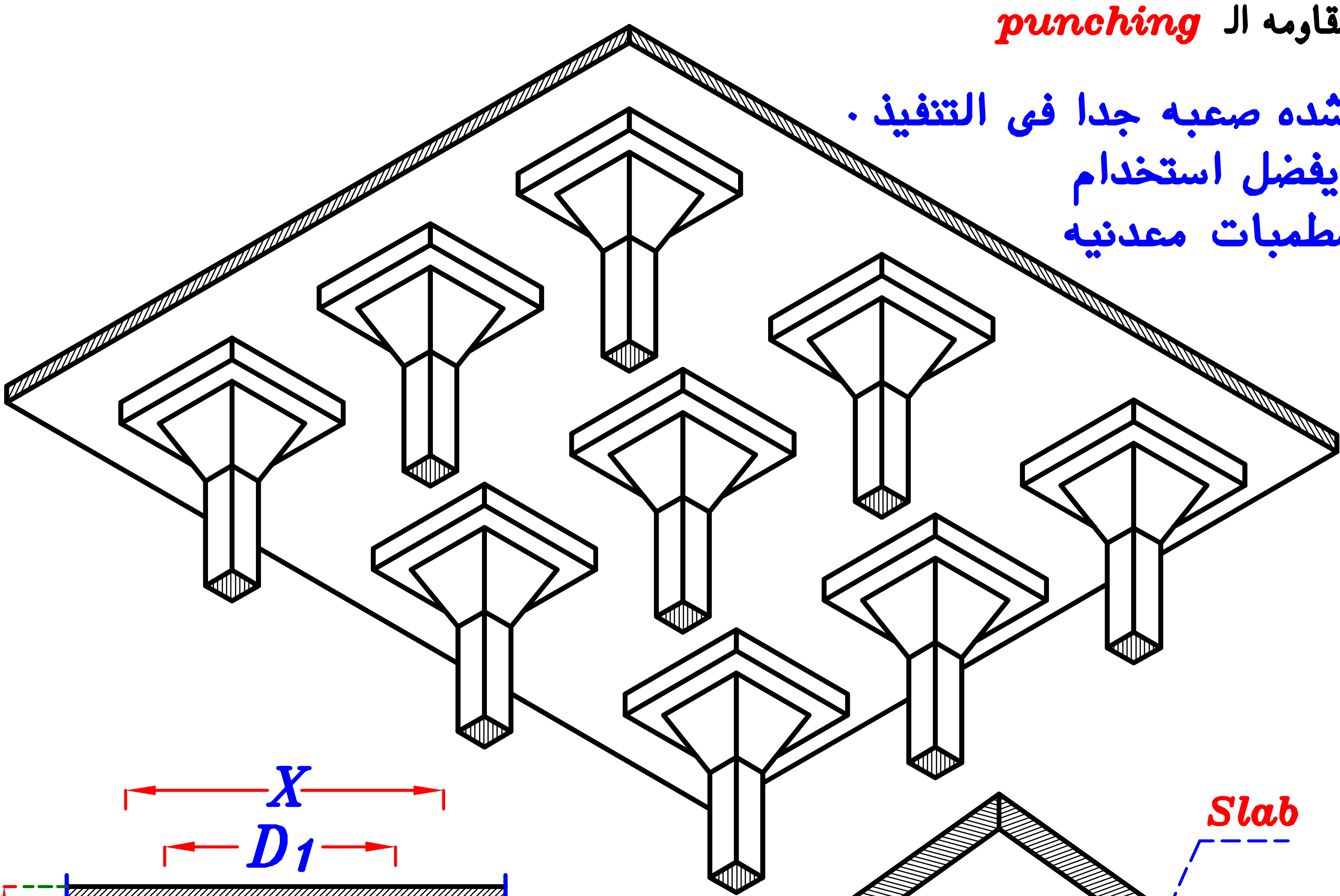
(Used to avoid punching and resist (-Ve) moments)

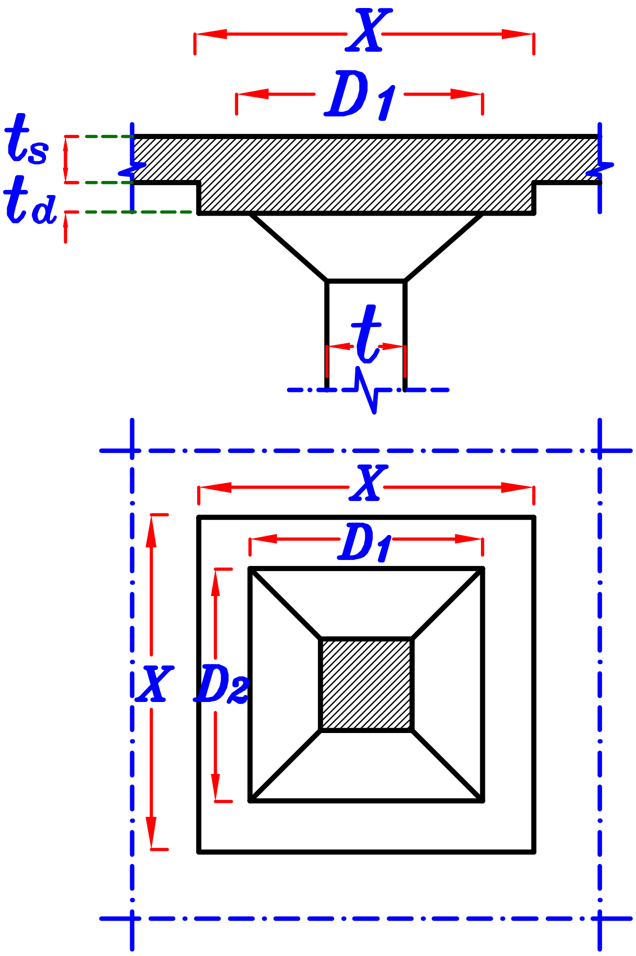
للاحمال العاليه و البحور الكبيره مثل الكبارى و الجراجات

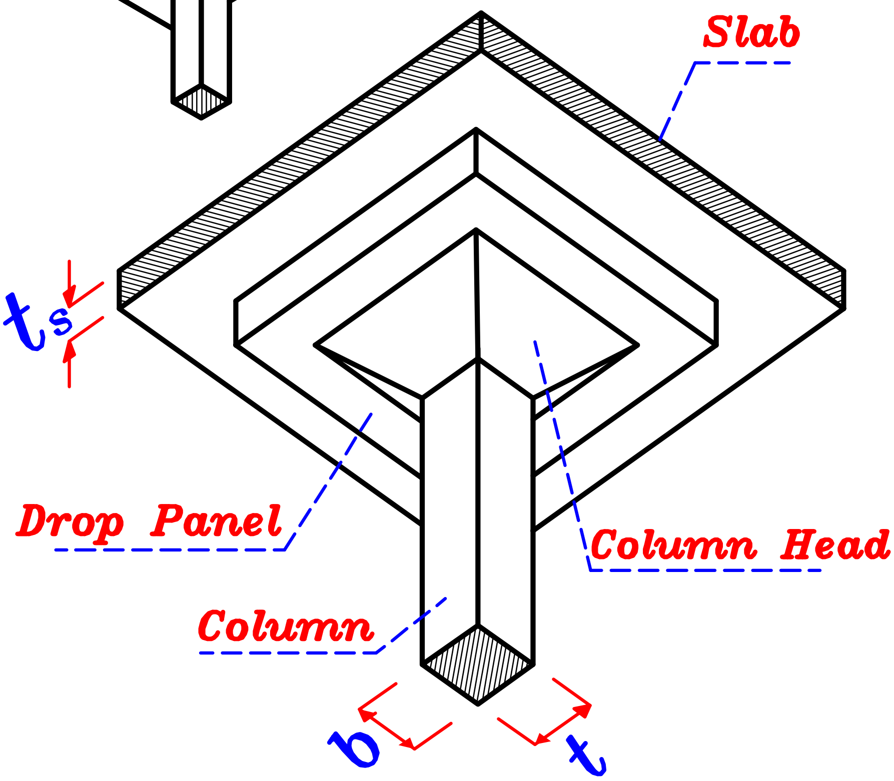
يتم عمل drop panel لمقاومه ال moments (-Ve) و عمل ال Column Head

لمقاومه ال punching

الشده صعبه جدا فى التنفيذ .
و يفضل استخدام
اسطمبات معدنيه

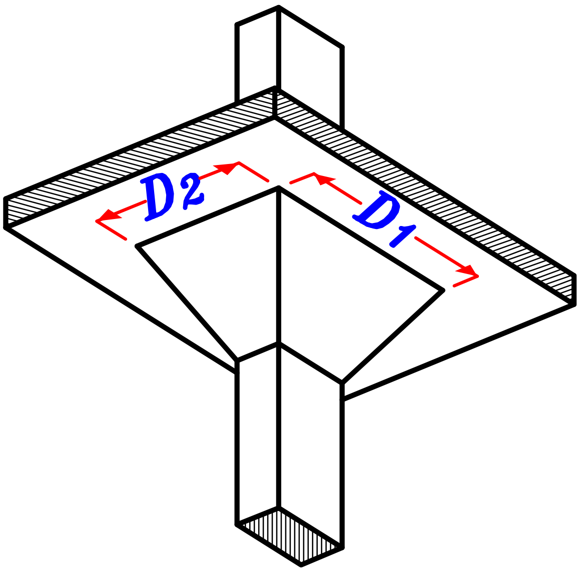




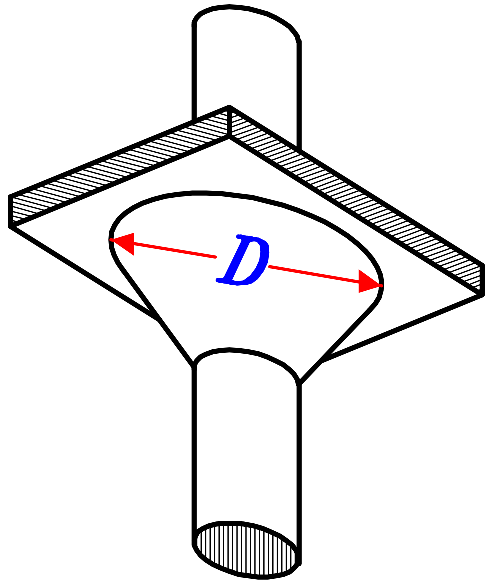


Dimensions of Column Head.

نستخدم ال ***Column Head*** عندما يكون ال ***Punching Stress*** كبير على البلاطه .



الأعمده المستطيله

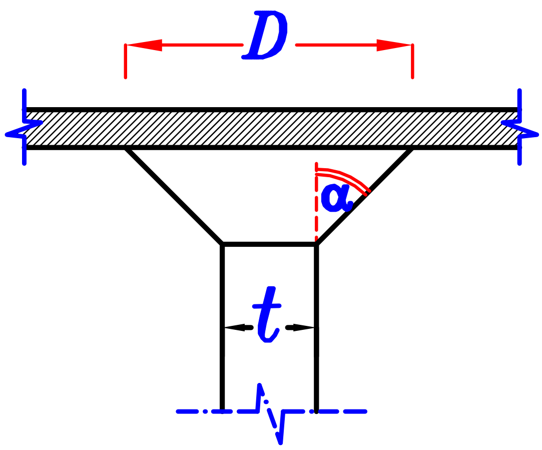


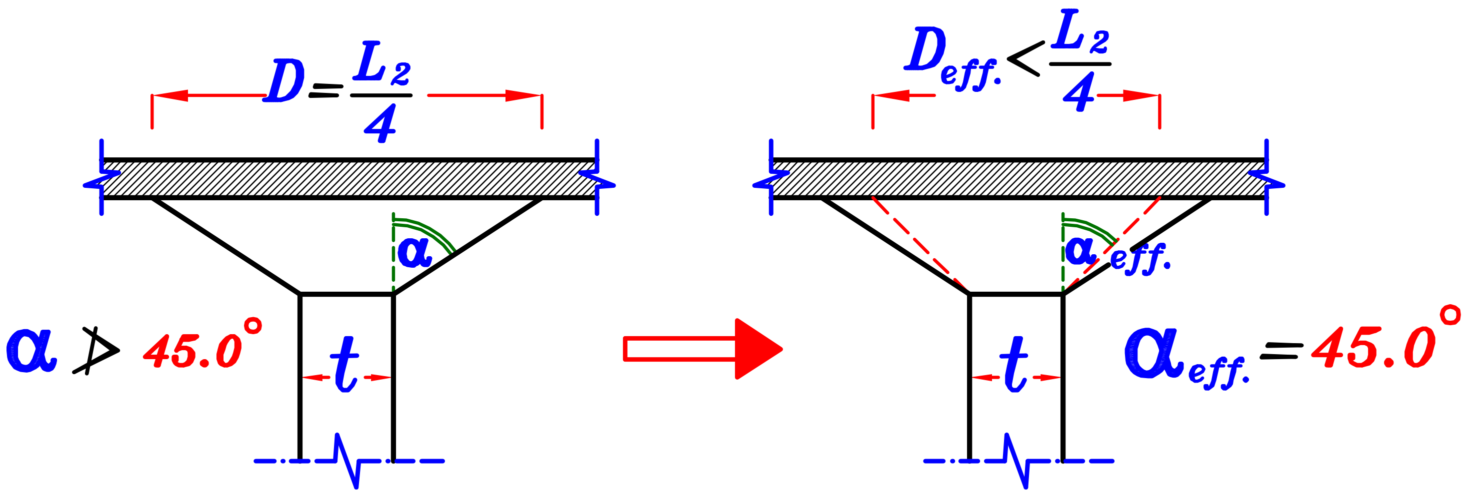
الأعمده الدائريه

$$\alpha \ngtr 45.0^\circ$$

$$D \ngtr \frac{L_2}{4}$$

L_2 هو العرض الاصغر للباكيه

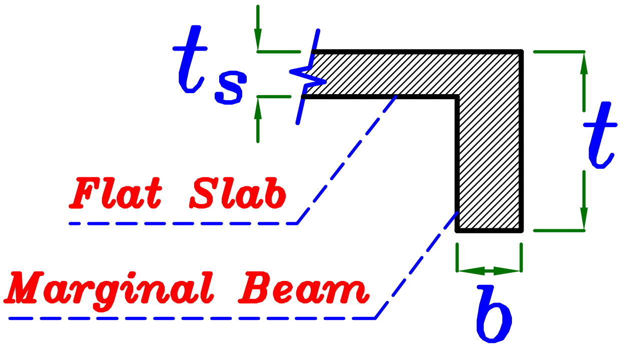




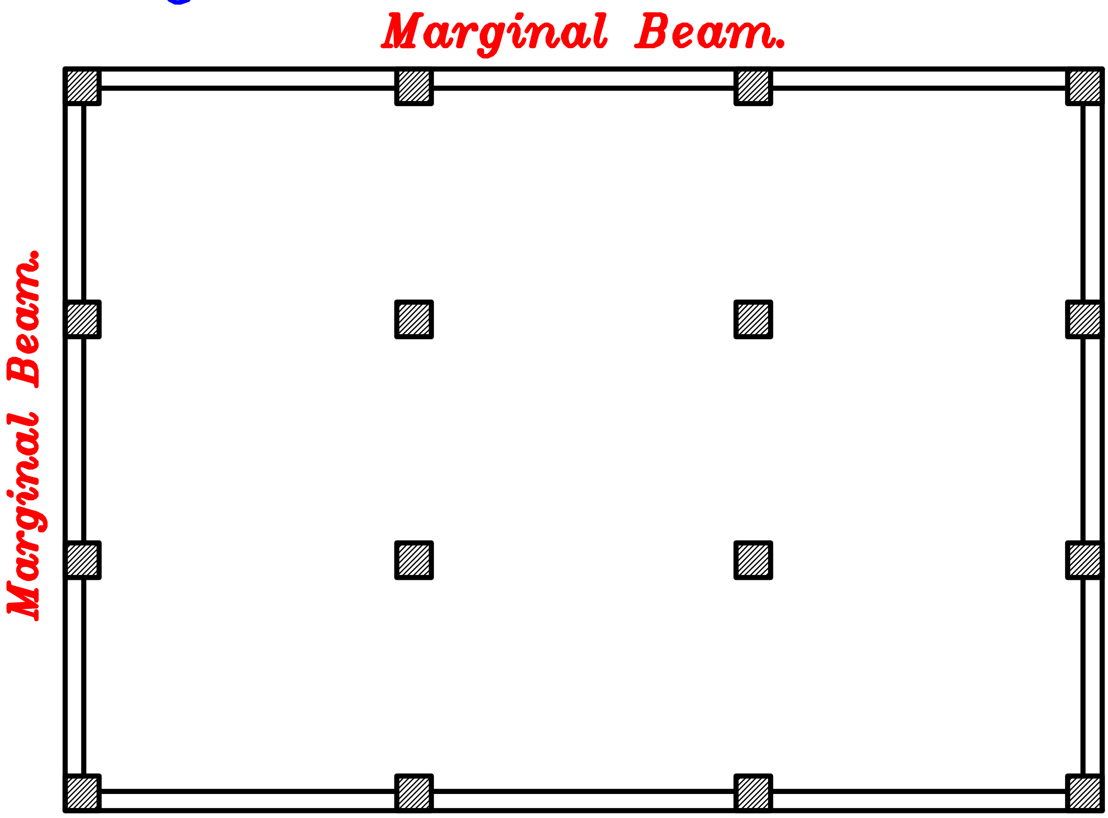
$$IF \quad \alpha > 45.0^\circ \xrightarrow{use} \alpha_{eff.} = 45.0^\circ \xrightarrow{Calculate} D_{eff.}$$

Marginal Beam.

هى عباره عن كمره تكون على أطراف البلاطه الخارجيه .
و ممكن وضع هذه الكمره أو ترك البلاطه بدونها .
و لكى نضمن أن تعمل هذه الكمره على حمل البلاطة و ليست محموله عليها
يجب أن تكون الـ ***Stiffness*** للكمره أكبر بكثير من البلاطه .



لذا يجب أن تكون $\boxed{t \geqslant 3\, t_s}$

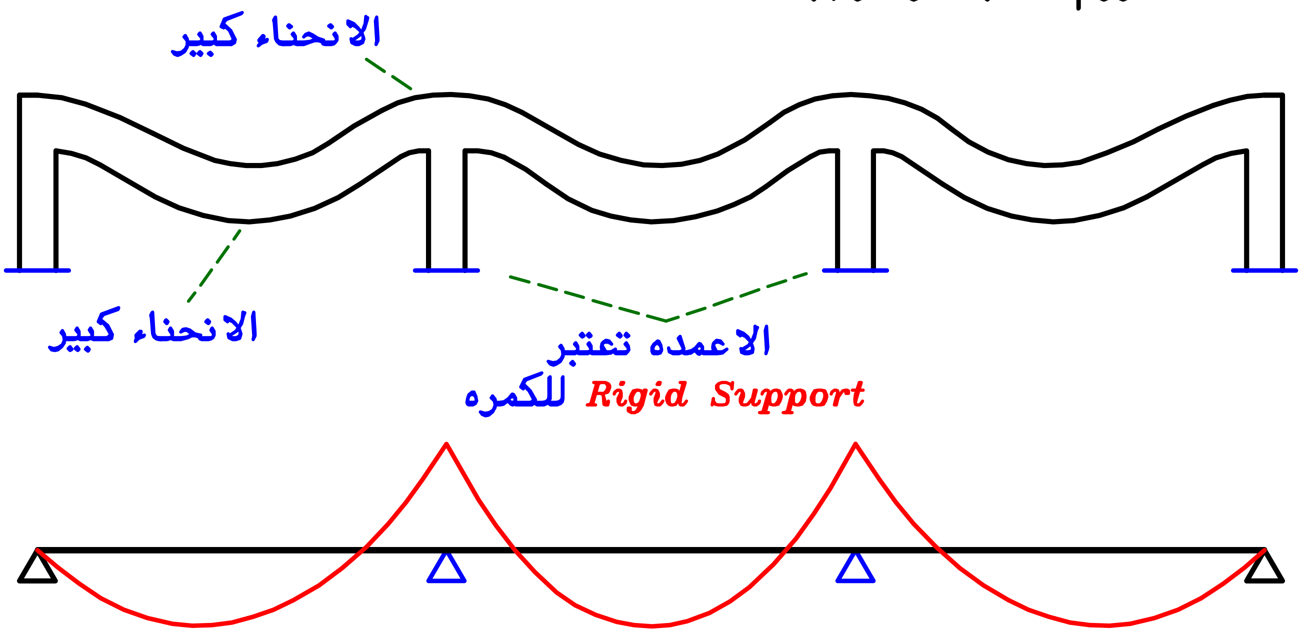


فوائد الـ *Marginal Beam* :

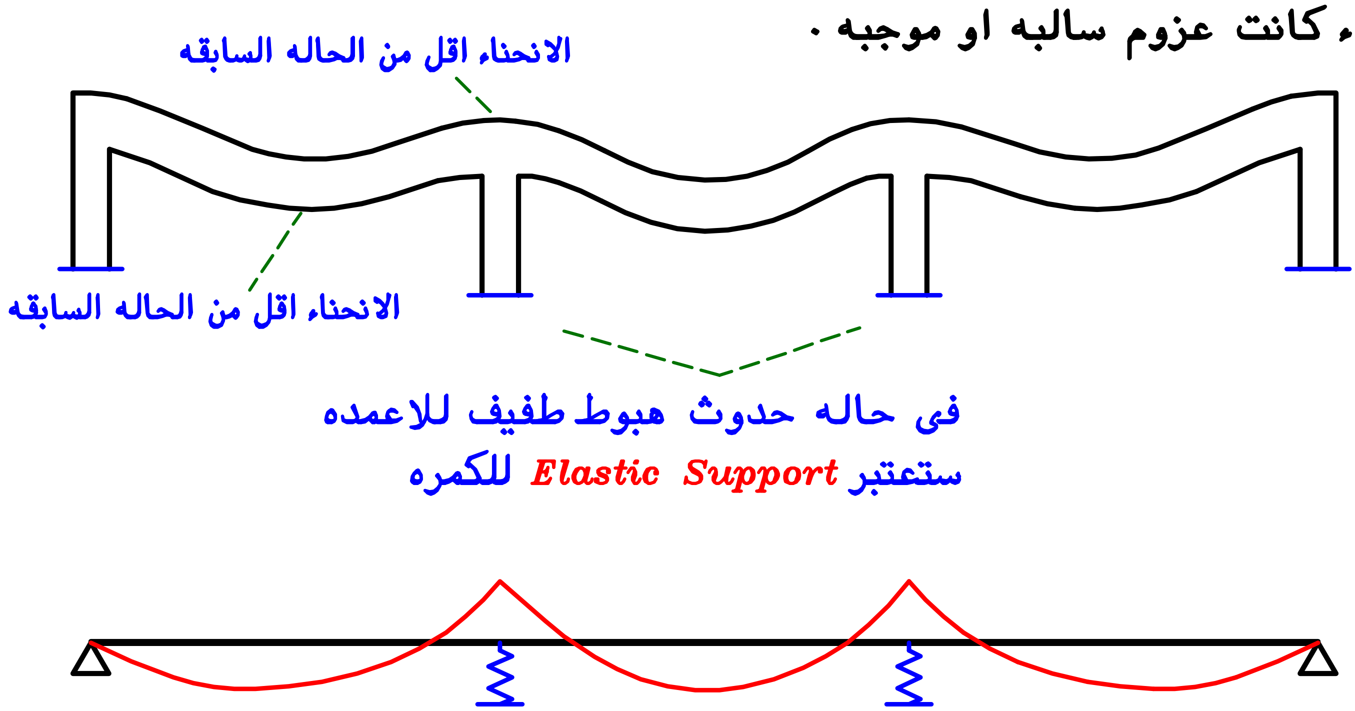
١- تحزيم المبنى لمقاومة الرياح و الزلازل .
٢- تقوية أطراف البلاطه .
٣- حمل حوائط الواجهه .

Rigid Support & Elastic Support.

المقصود بال **Rigid Support** هو ركيزه لن يحدث لها أى هبوط.
أى عند حدوث العزوم ستكون العزوم على العنصر المحمول كبيره
سواء كانت عزوم سالبه او موجبه .



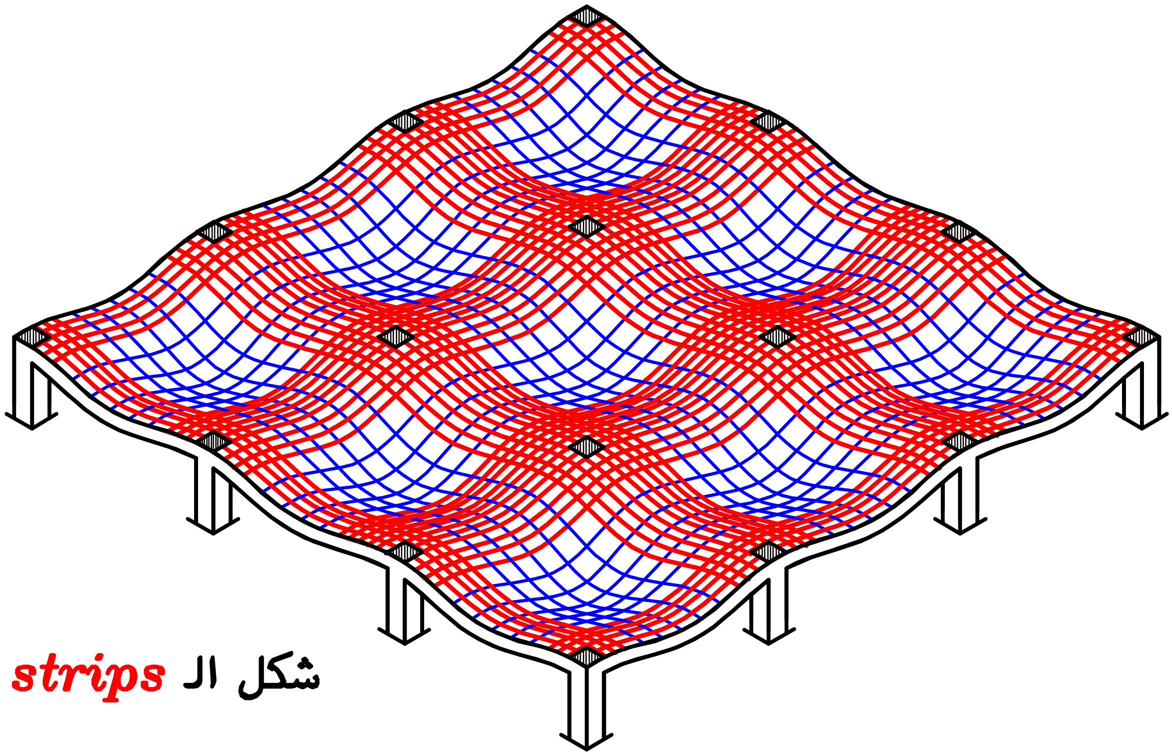
المقصود بال **Elastic Support** هو ركيزه ممكن ان يحدث لها بعض الهبوط الطفيف.
أى عند حدوث العزوم ستكون العزوم على العنصر المحمول اصغر من الحالة السابقه
سواء كانت عزوم سالبه او موجبه .



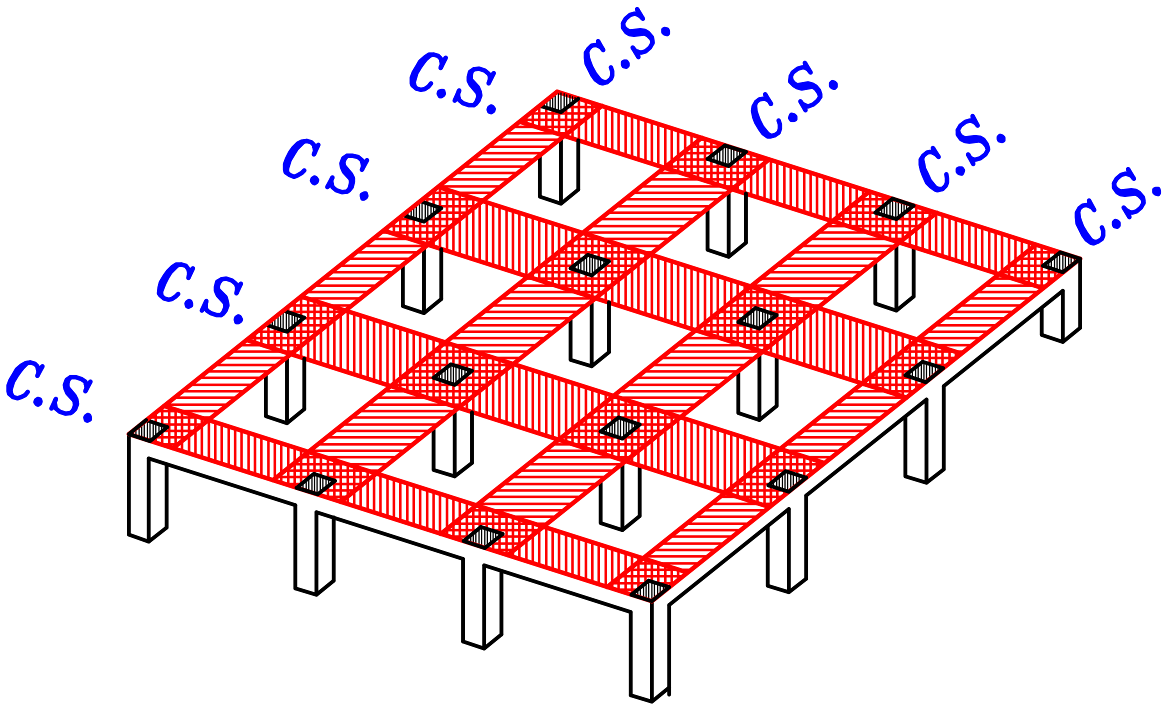
Concept of analysis For Flat Slab.

تعتمد فكره تحليل البلاطات ال Flat Slab الى توزيع عزوم الباكيه كلها على نوعين من الشرائح
النوع الاول هى شريحه العمود Column strip و تكون محموله على الاعمده .
النوع الثانى هى شريحه الوسط Field strip و تكون محموله على ال Column strip العموديه عليها . أى كأنها محموله على كمرات مدفونه .

ال Column strips عباره عن شرائح محموله مباشره على الاعمده
و تكون ال stiffness لهذه الشرائح اكبر من شرائح ال Field strips
لاننا سنضع بها كميه حديد اكبر و لانها محموله على اعمده اى على Rigid Supports



شكل ال Column strips

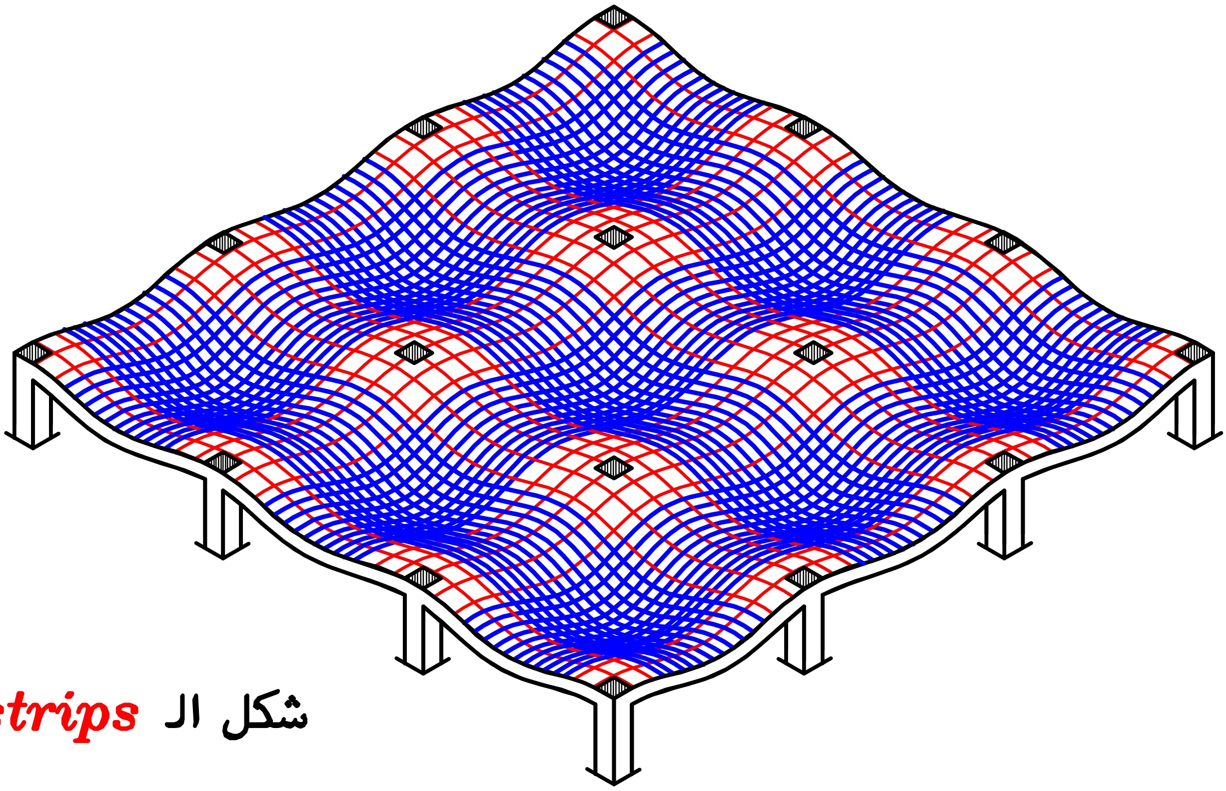


ال *Field strips* عباره عن شرائح محموله على شرائح ال *Column strips* العموديه عليها .

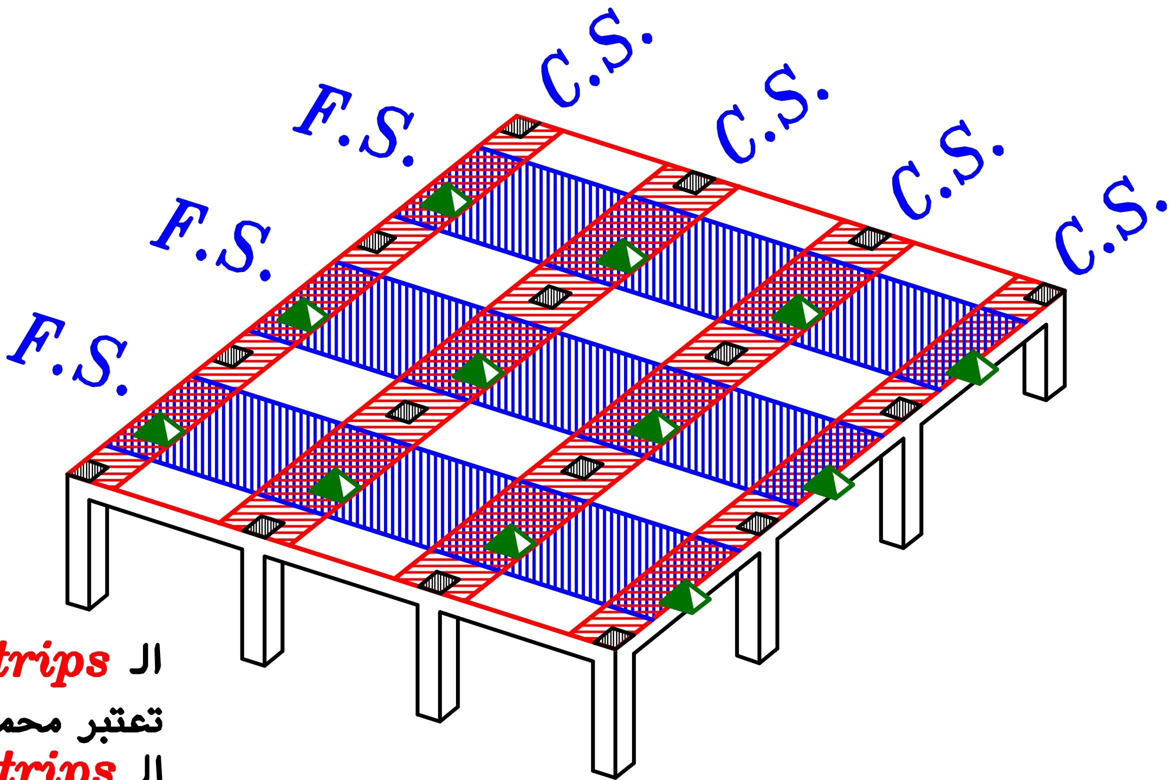
و ذلك لان ال *stiffness* لل *Field strips* اقل من ال *Column strips* العموديه عليها

لانها تعتبر محموله على *Elastic Supports* بينما ال *Column strips*

محموله على *Rigid Supports*



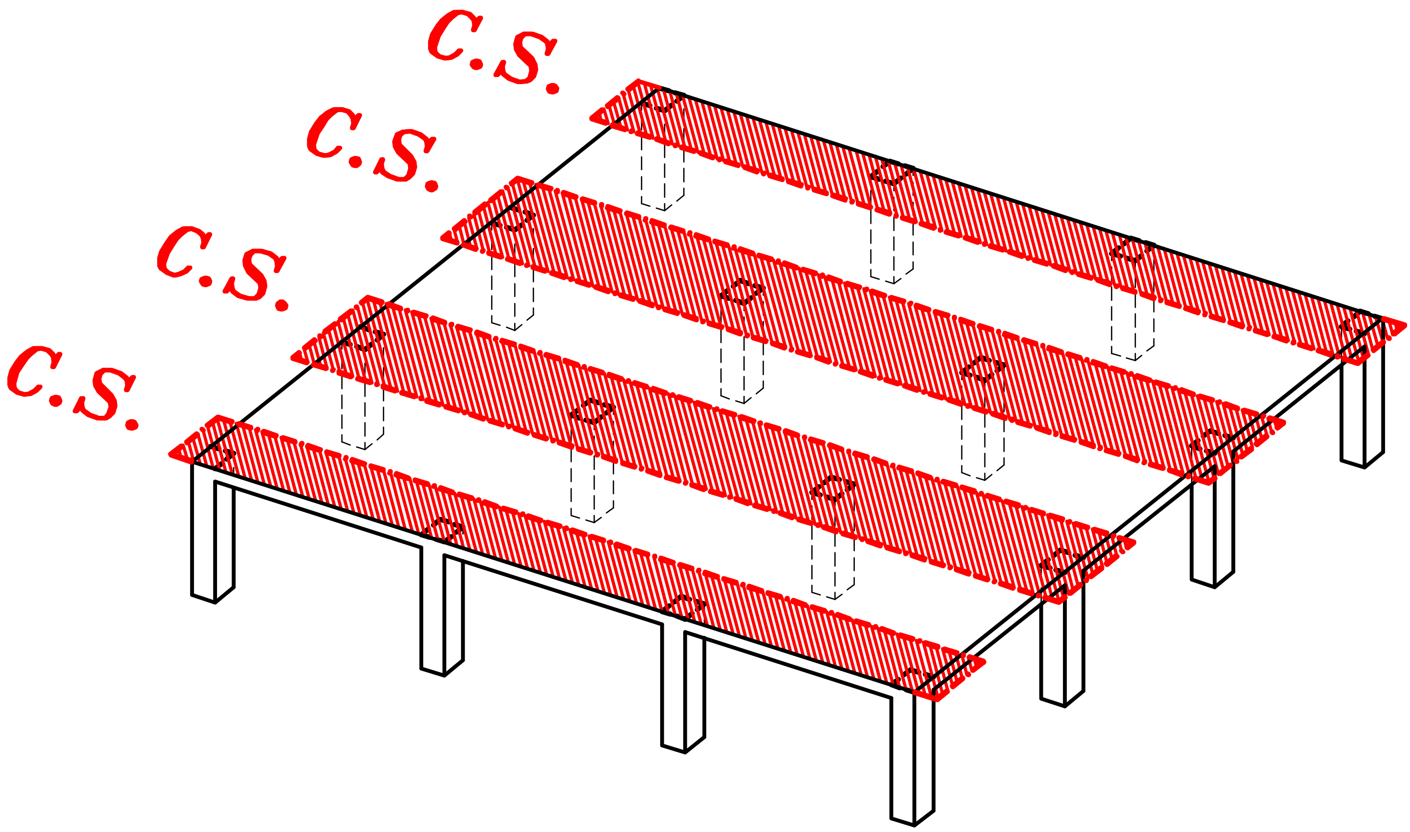
شكل ال *Field strips*

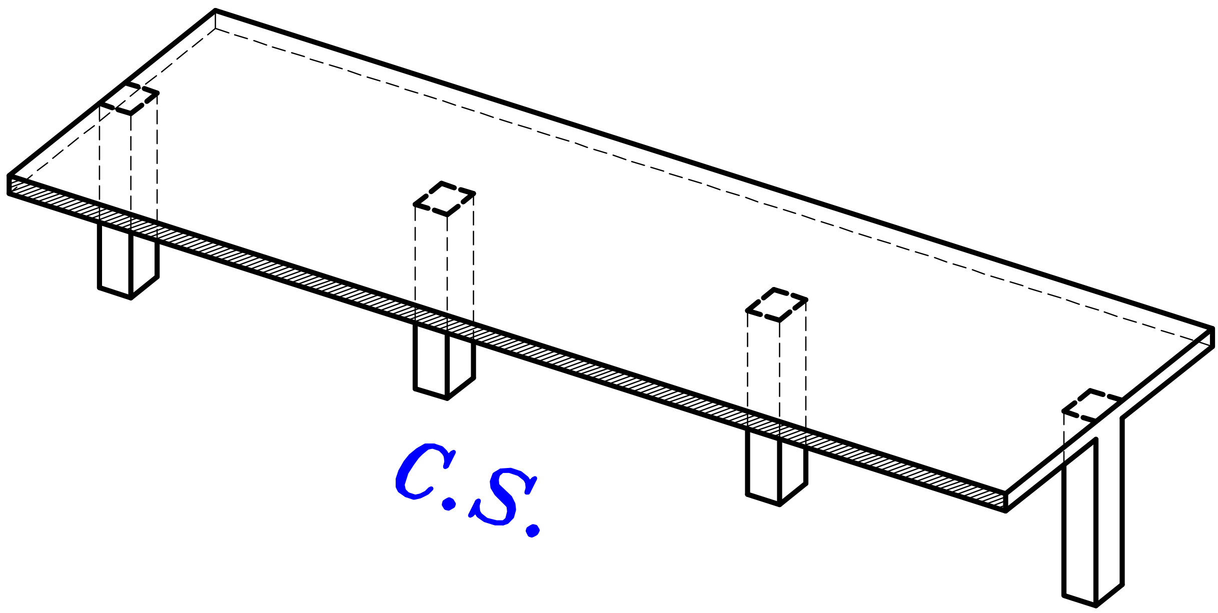


ال *Field strips*
تعتبر محموله على
ال *Column strips*
العموديه عليها

Column Strip. (C.S.)

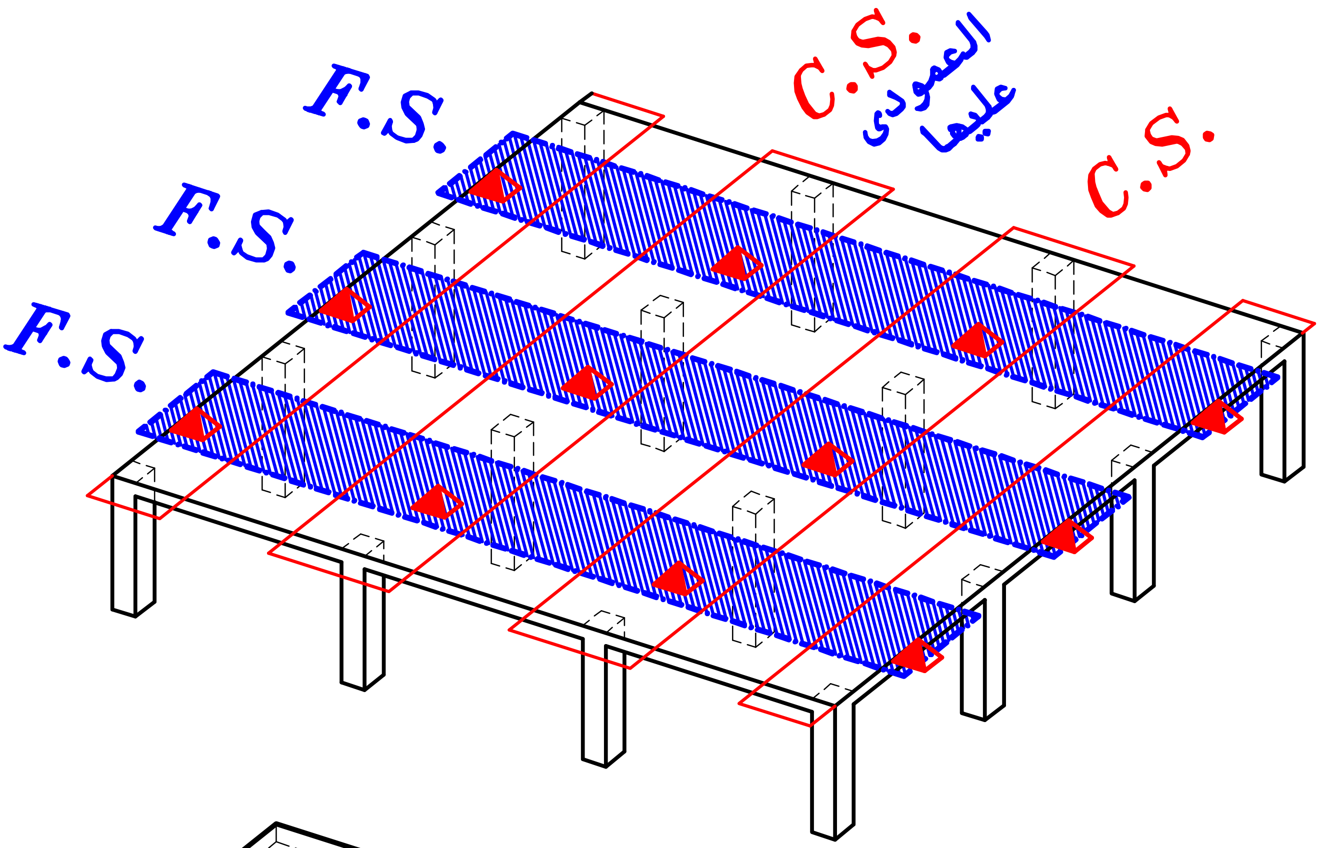
شريحه العمود *Column strip* تكون محموله على الاعمده .

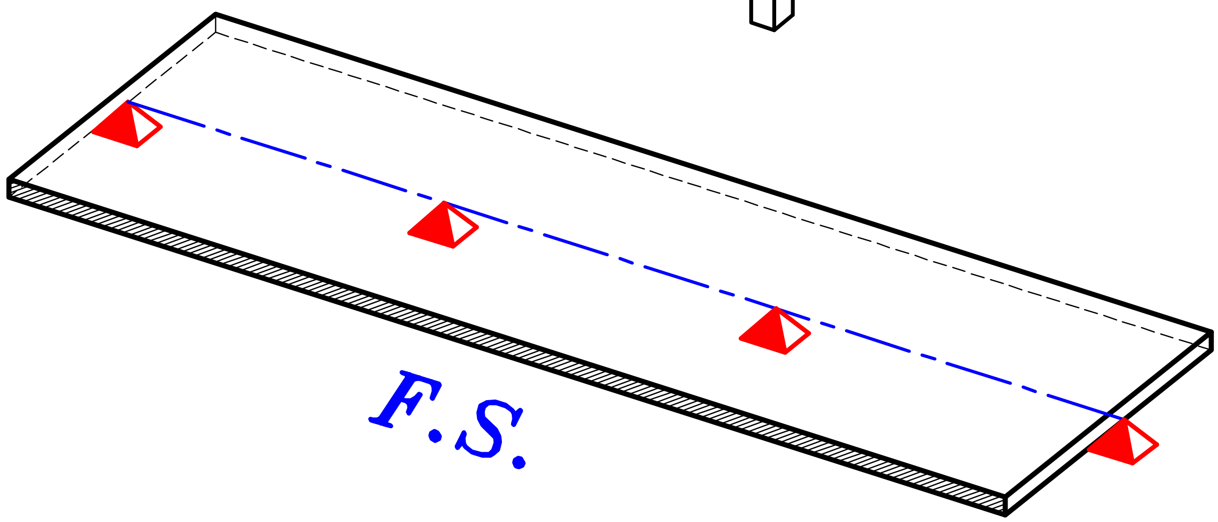




Field Strip. (F.S.)

شريحه الوسط Field strip تكون محموله على ال Column strip العموديه عليها.





لان ال Column strip محموله على Rigid support و هى الاعمده .

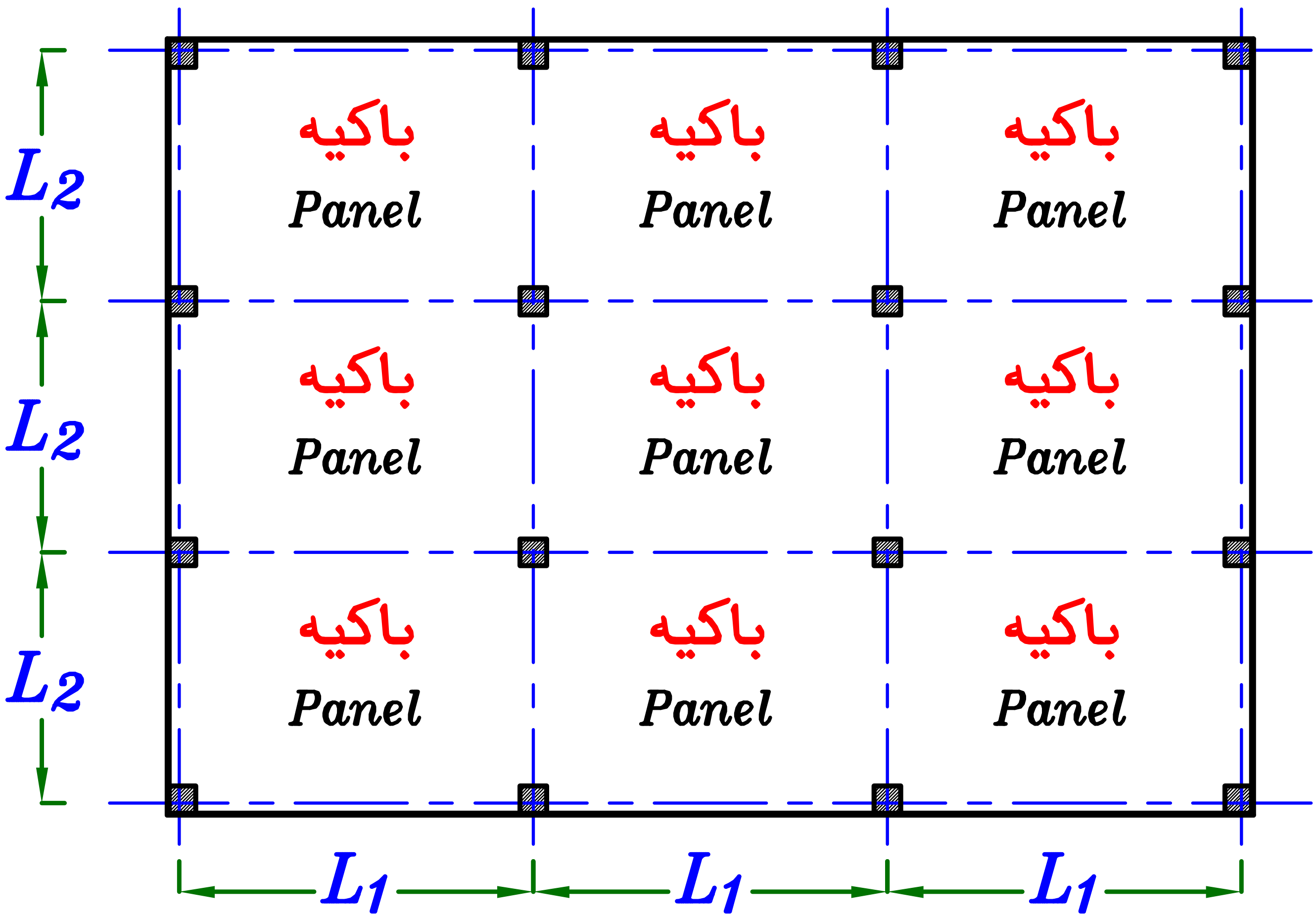
و لان ال Field strip محموله على elastic support و هى ال Column strips العموديه عليها

فتكون العزوم المتولده على ال column strip أكبر من ال Field strip

Width of the Strips تحديد عرض الشرائح

تم الاتفاق فى التصميم على فرض قيم لعرض شرائح الـ **Column strips** و شرائح الـ **Field Strips** ثم حساب الاحمال و العزوم على هذه الشرائح و تصميمها و وضع الحديد بها .

و لان حديد الـ **Column strip** الناتج عن التصميم سيكون اكبر من الـ **Field Strip** و لان هذا الحديد سيوضع فى العرض الذى فرضناه للـ **Column strip** اذا سنضمن فى الحقيقه ان هذا العرض سيعمل كله على انه **Column strip** لان فعليا ستكون الـ **Stiffness** له اكبر لان به كميه حديد اكبر .



الباكيه الواحده هى المساحه الواقعه بين ٤ أعمدة .

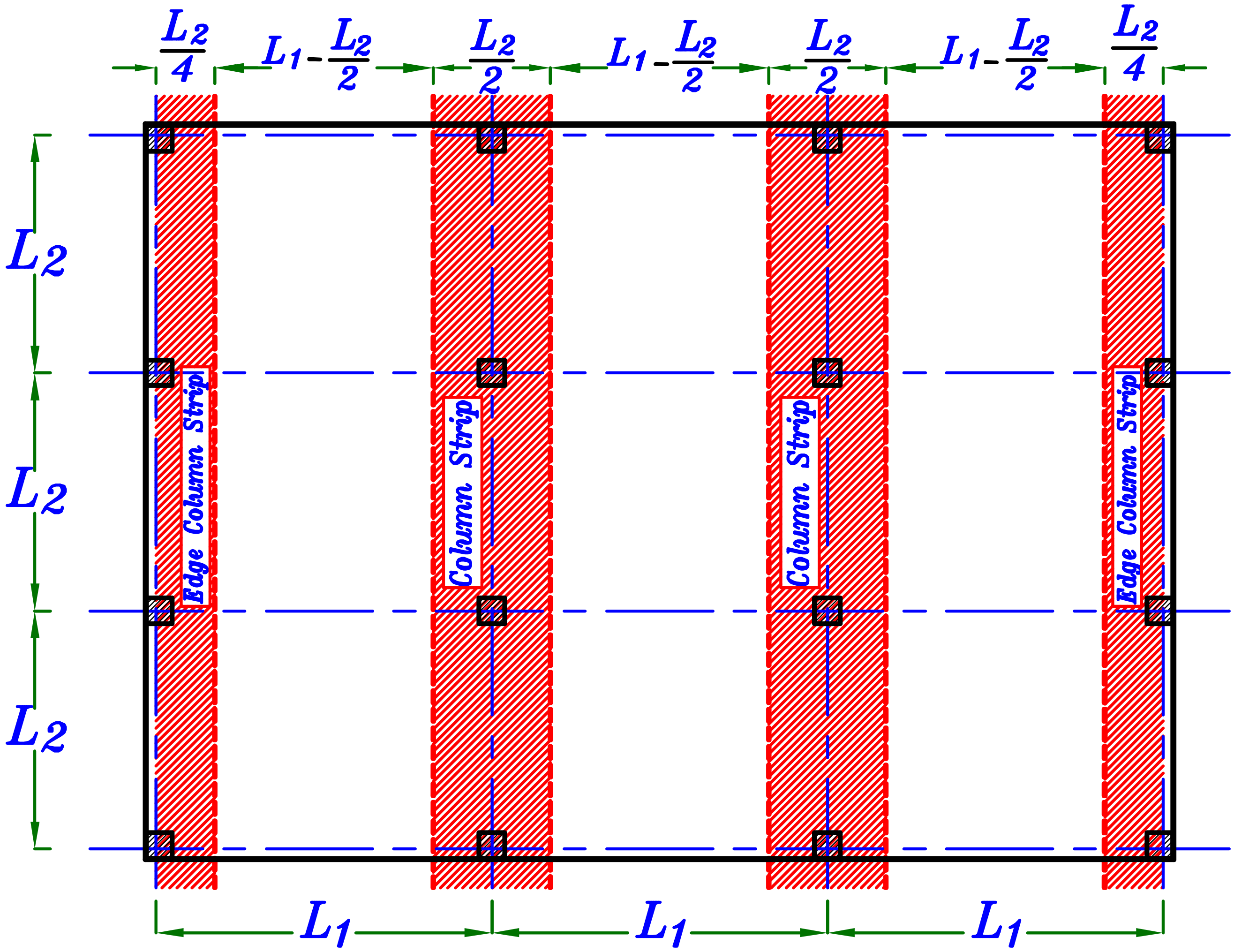
الطول الاكبر للباكيه يسمى L_1

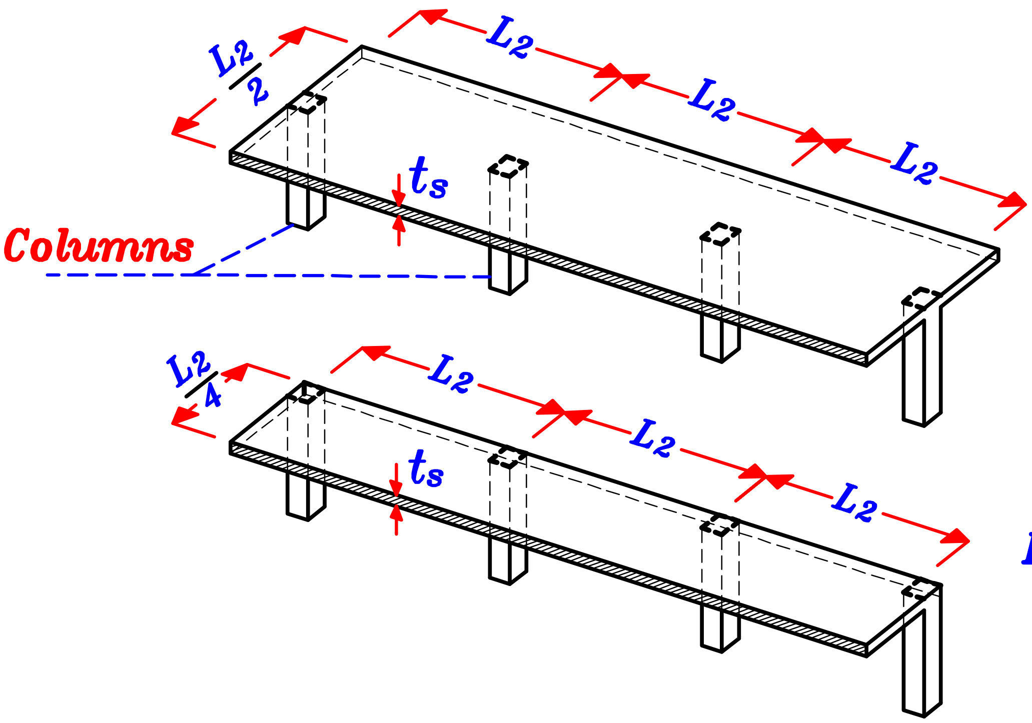
الطول الاصغر للباكيه يسمى L_2

Without Drop Panel. (Strips in Short Direction)

نفرض ان عرض ال Column Strip = $\frac{L_2}{2}$ (فى الاتجاهين)

اذا عرض ال Field Strip = باقى المسافه من C.L. to C.L.

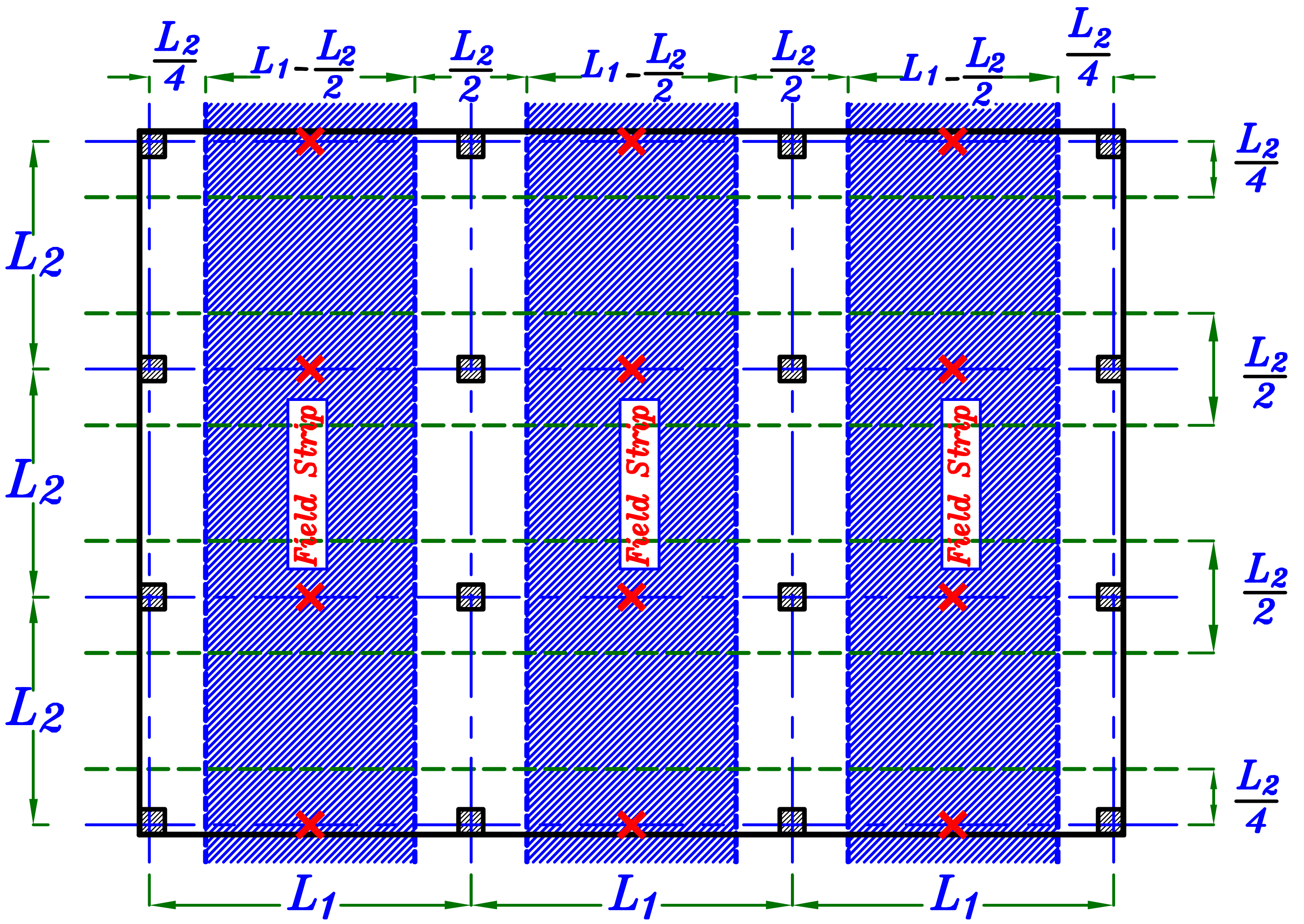


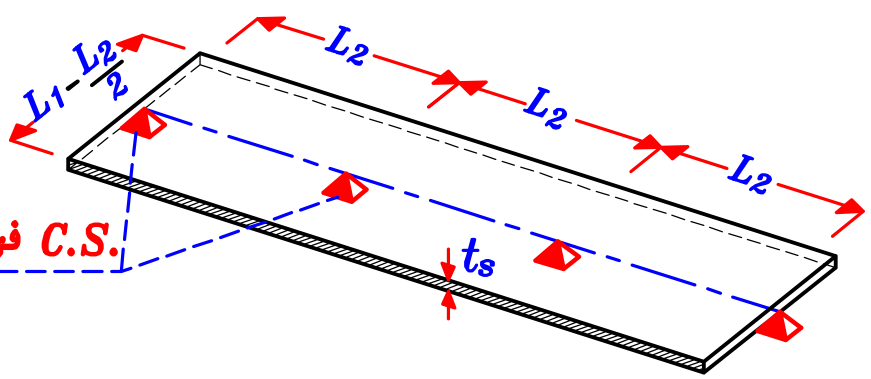


Without Drop Panel. (Strips in Short Direction)

نفرض ان عرض ال **Column Strip** = $\frac{L_2}{2}$ (فى الاتجاهين)

اذا عرض ال **Field Strip** = باقى المسافه من **C.L. to C.L.**

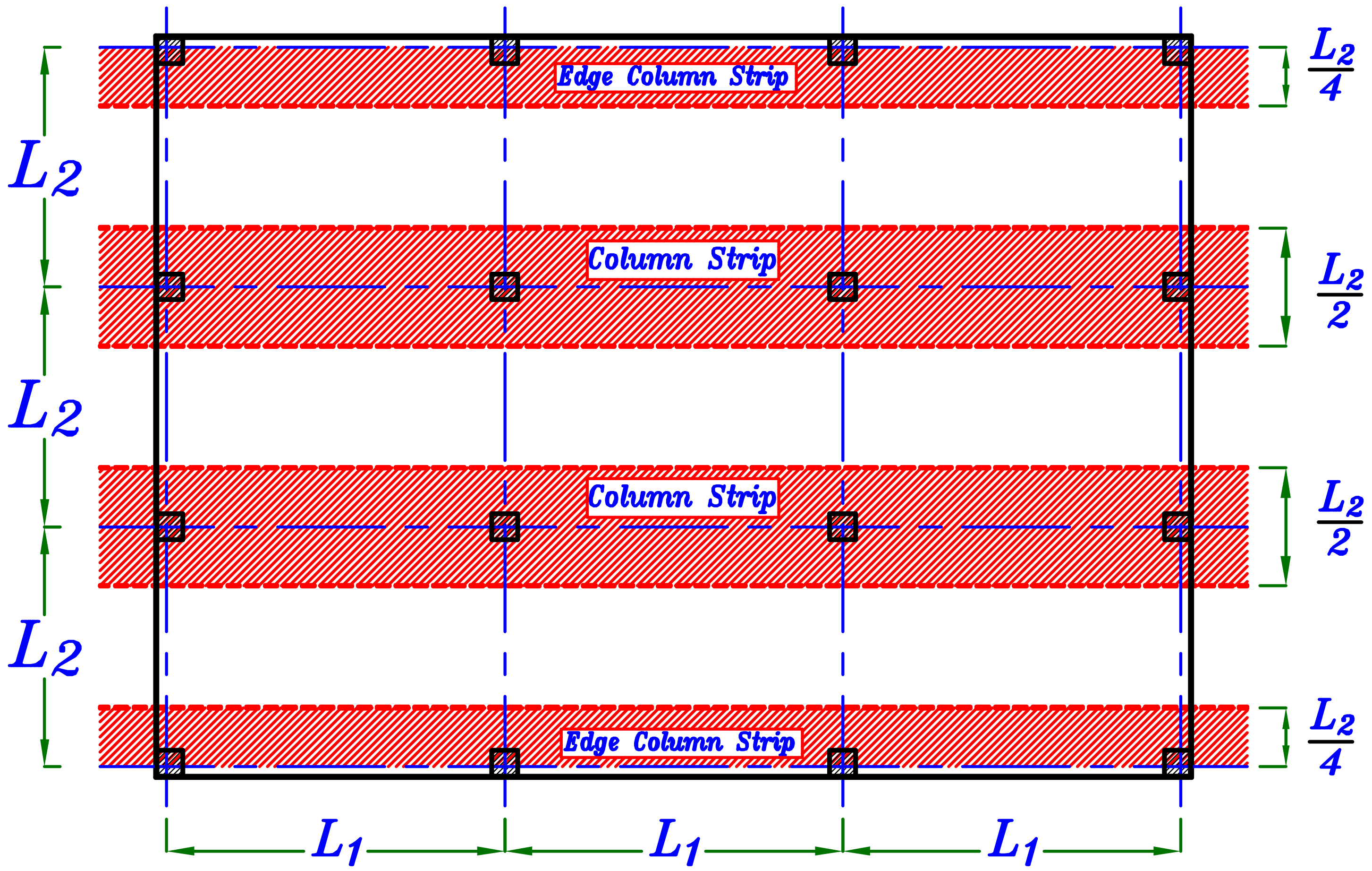


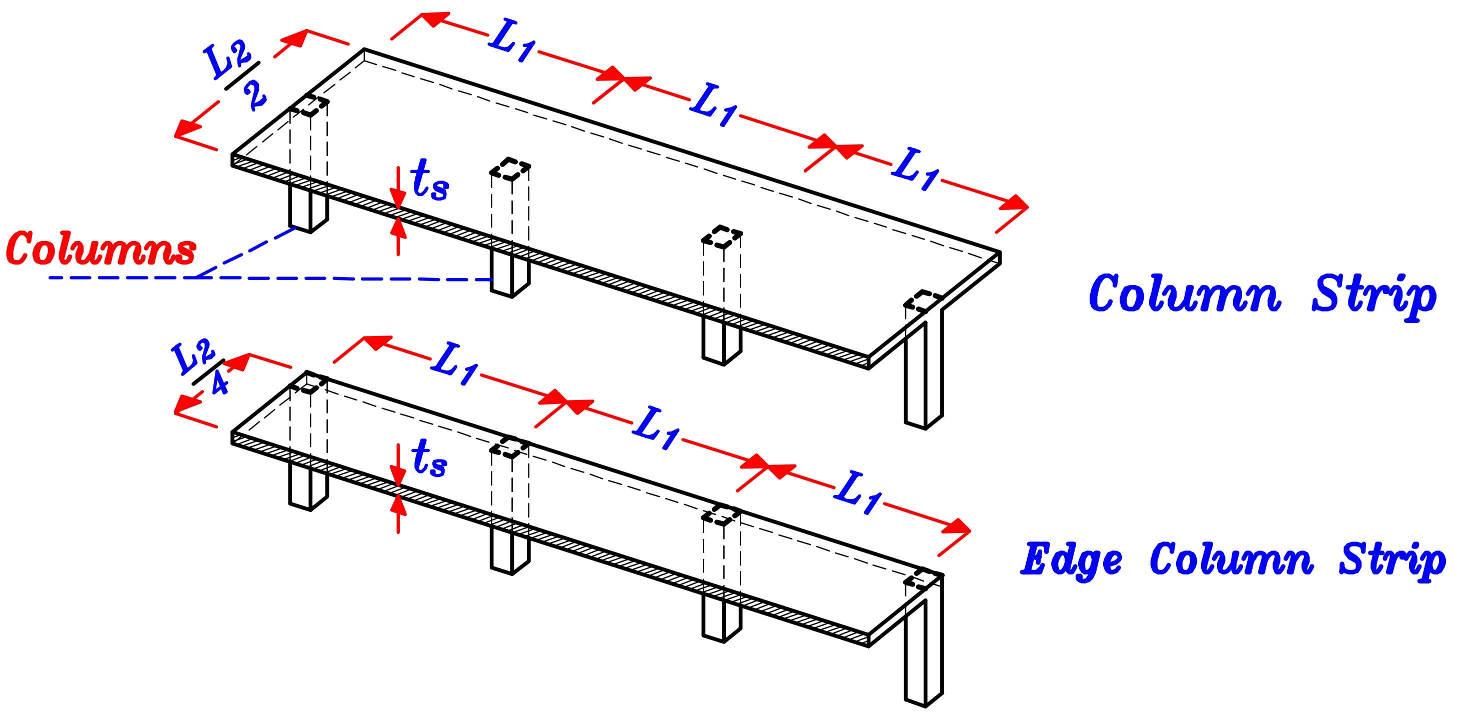


Without Drop Panel. (Strips in Long Direction)

نفرض ان عرض ال Column Strip = $\frac{L_2}{2}$ (فى الاتجاهين)

اذا عرض ال Field Strip = باقى المسافه من C.L. to C.L.

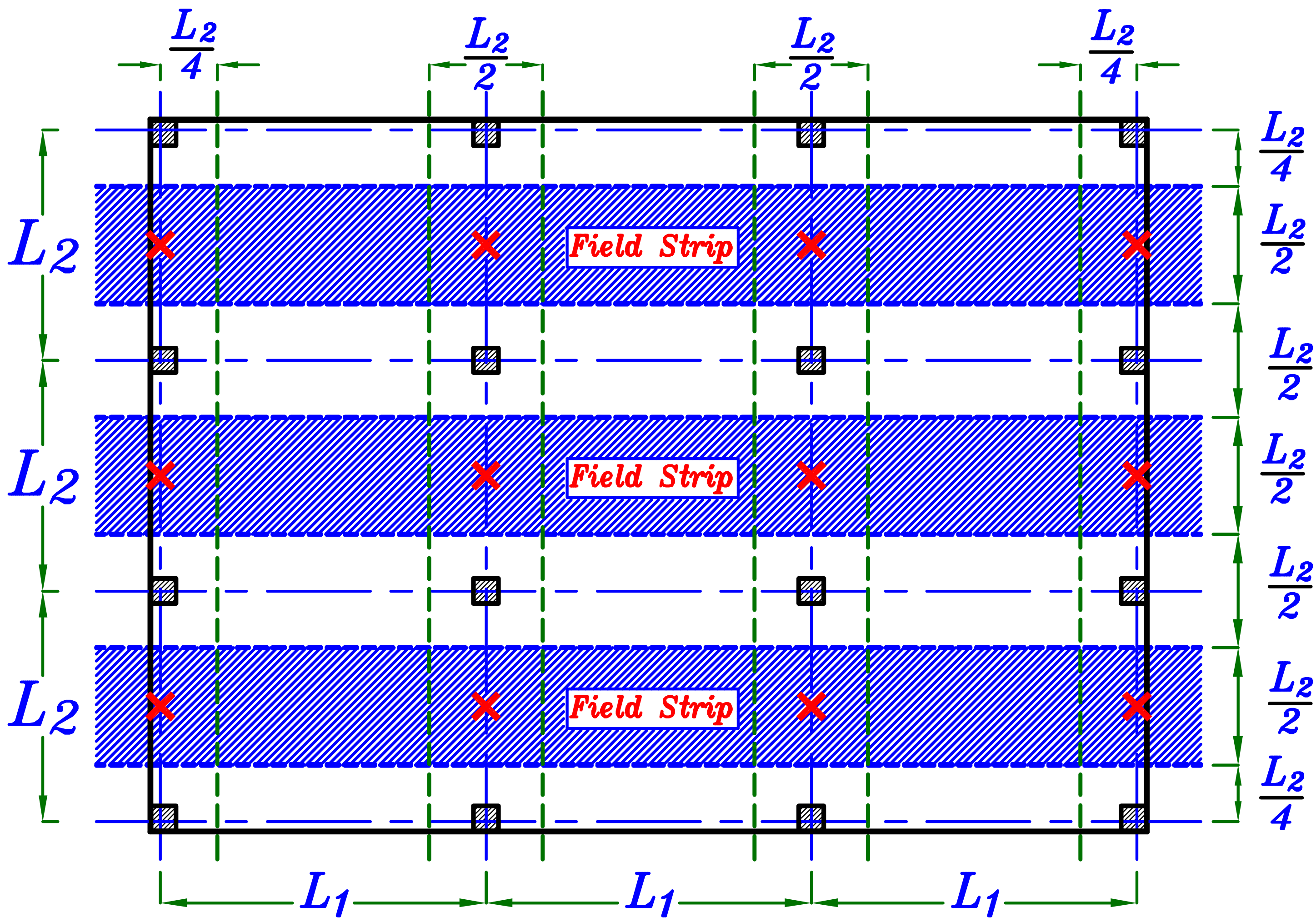


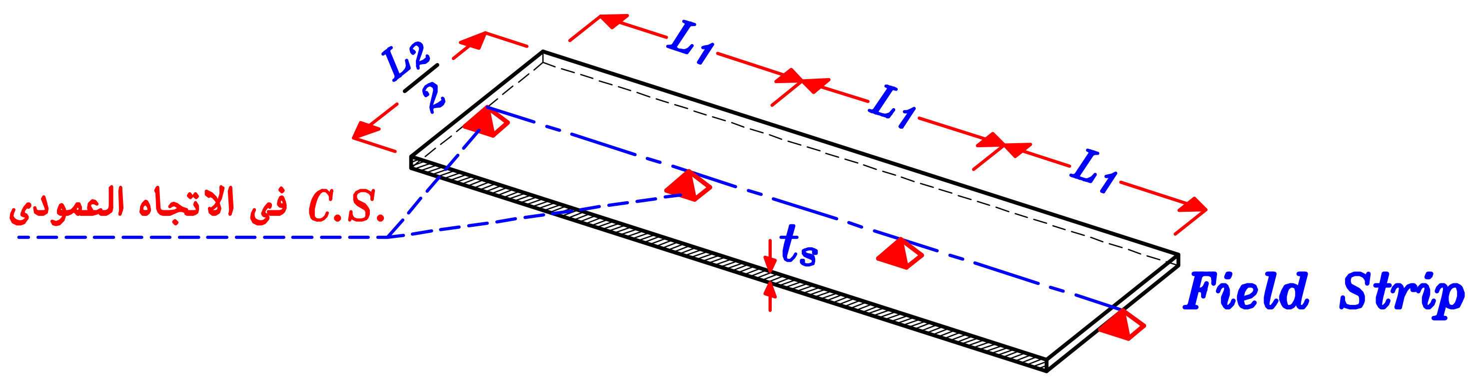


Without Drop Panel. (Strips in Long Direction)

نفرض ان عرض ال Column Strip = $\frac{L_2}{2}$ (فى الاتجاهين)

اذا عرض ال Field Strip = باقى المسافه من C.L. to C.L.

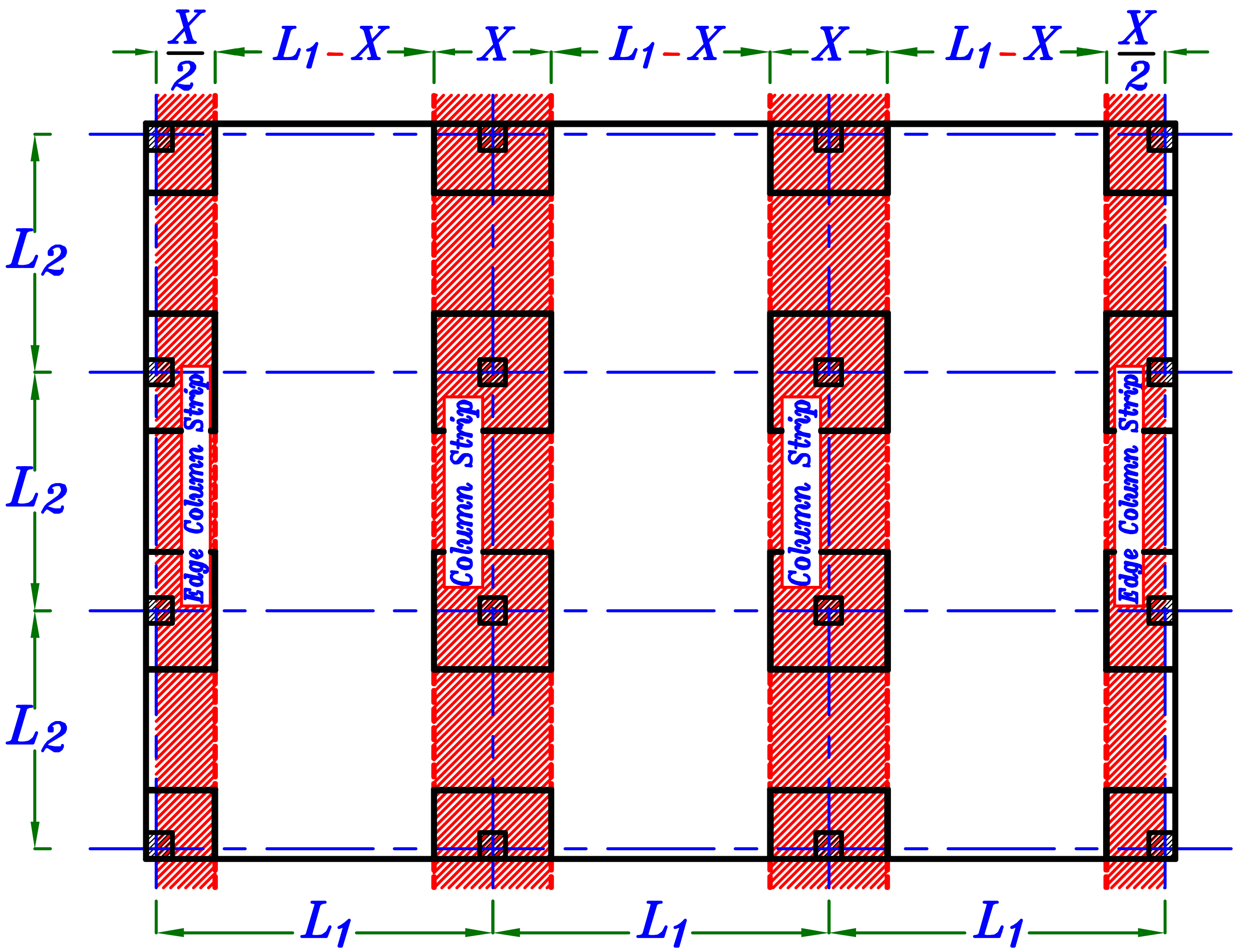


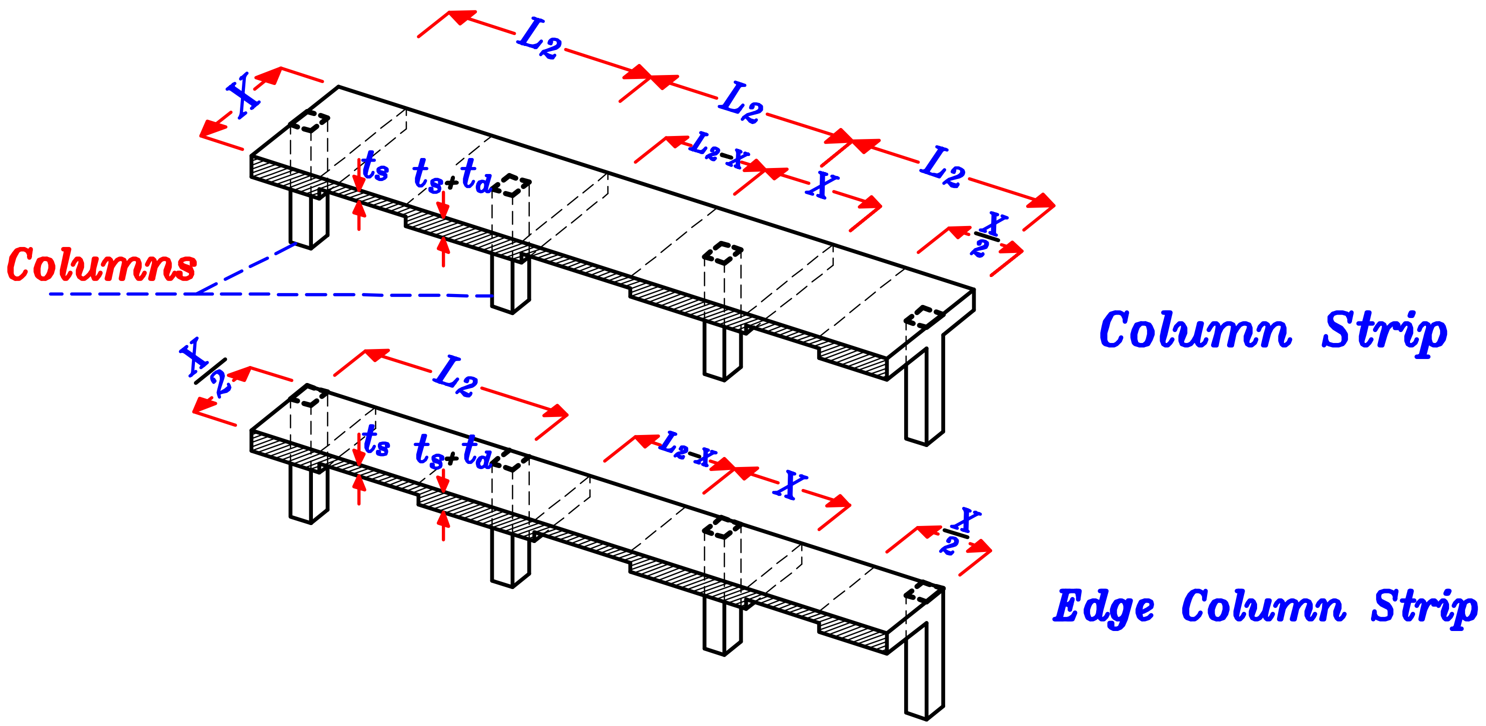


With Drop Panel. (Strips in Short Direction)

نفرض ان عرض ال Column Strip = X = عرض الdrop panel (فى الاتجاهين)

عرض ال Field Strip = باقى المسافه من C.L. to C.L.





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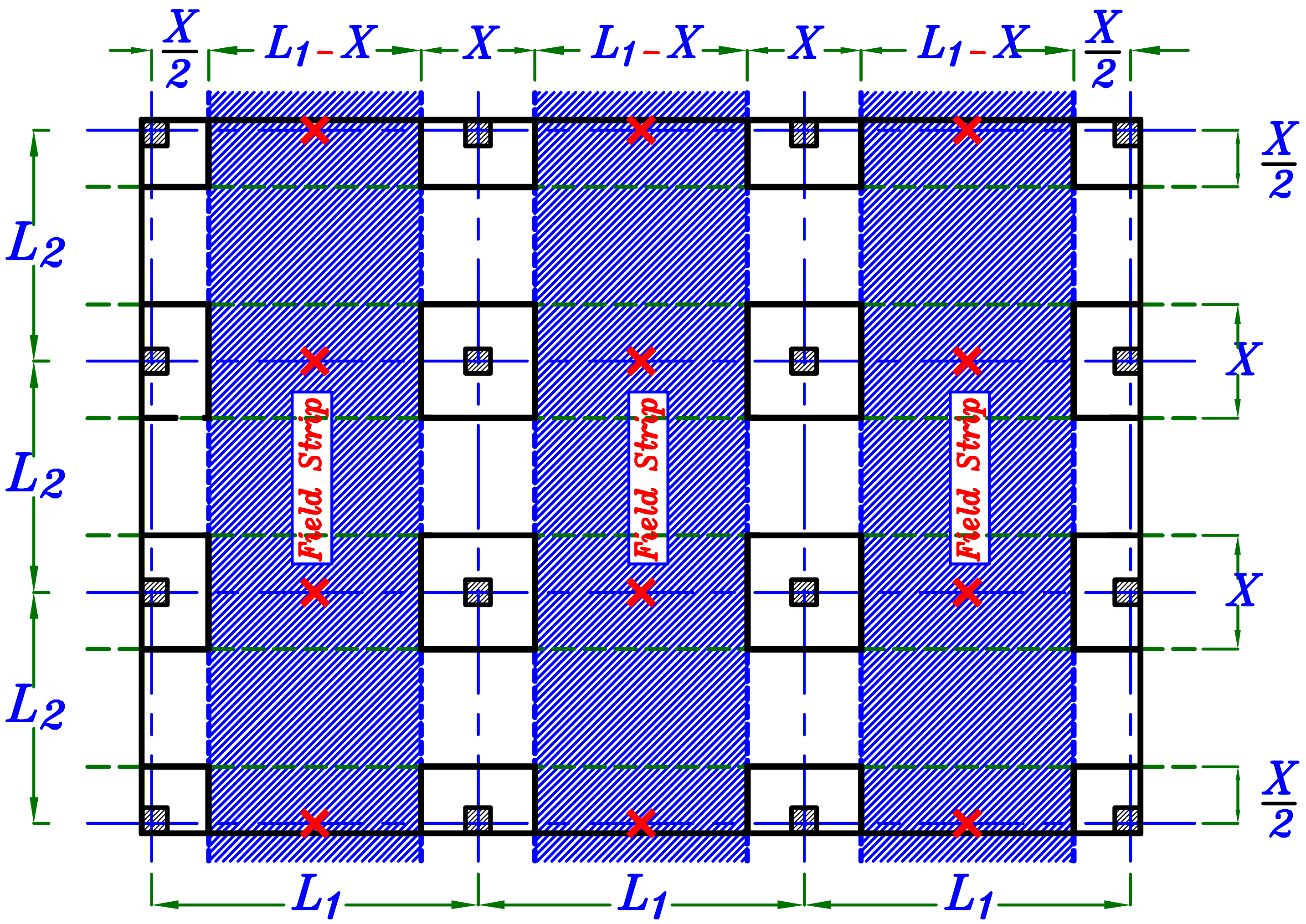
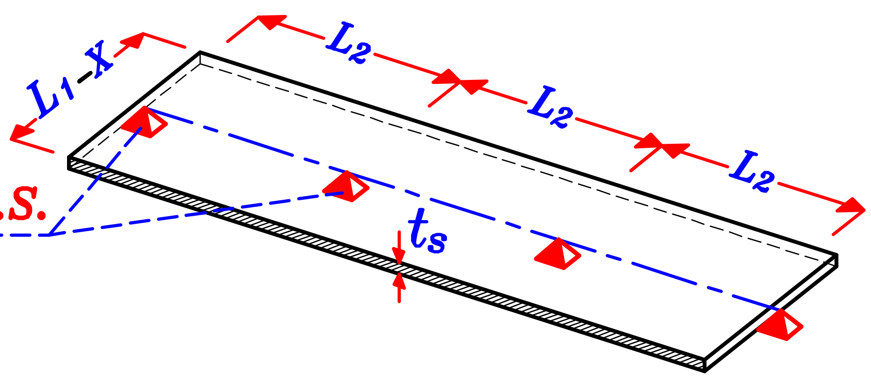
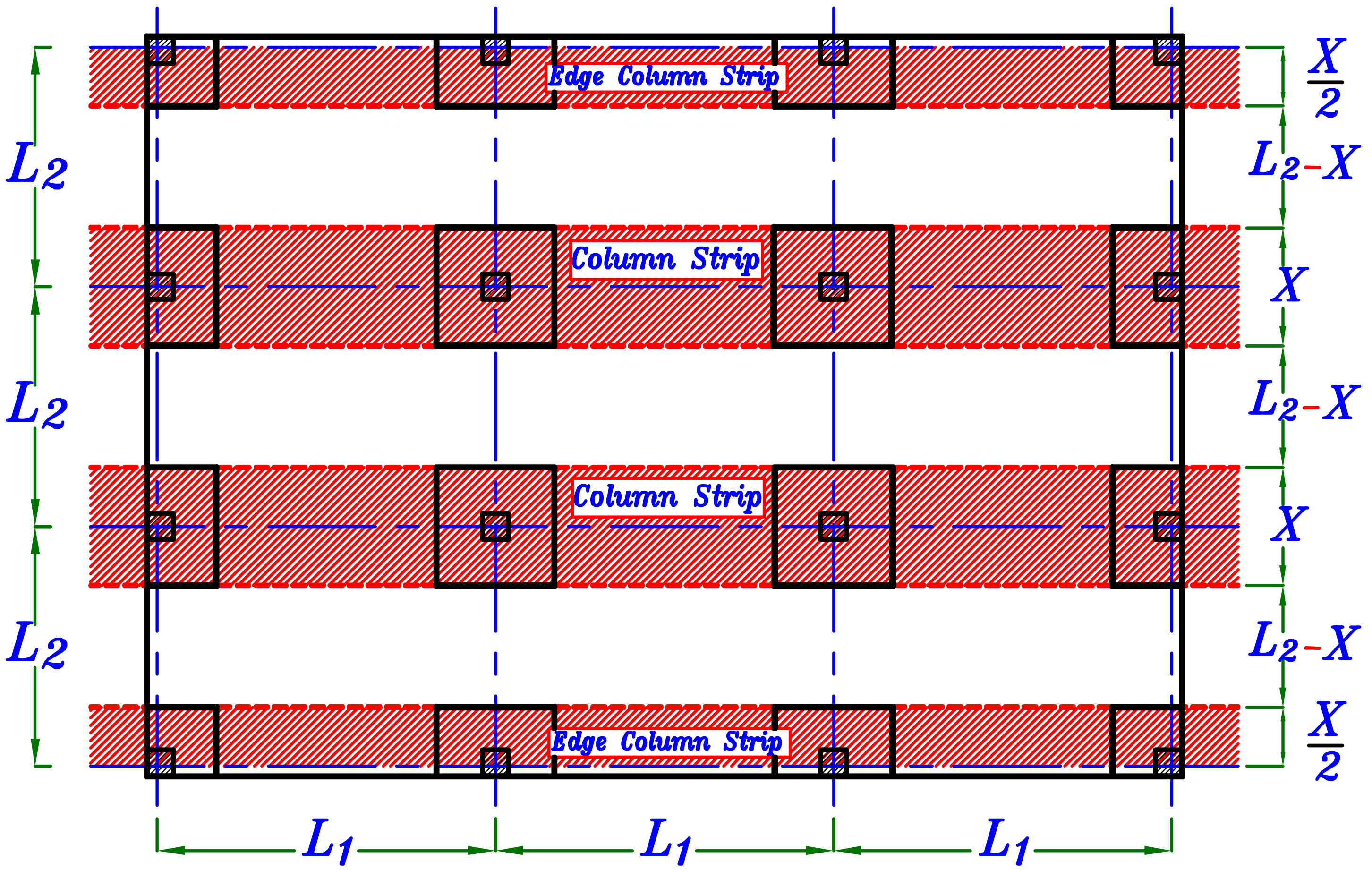


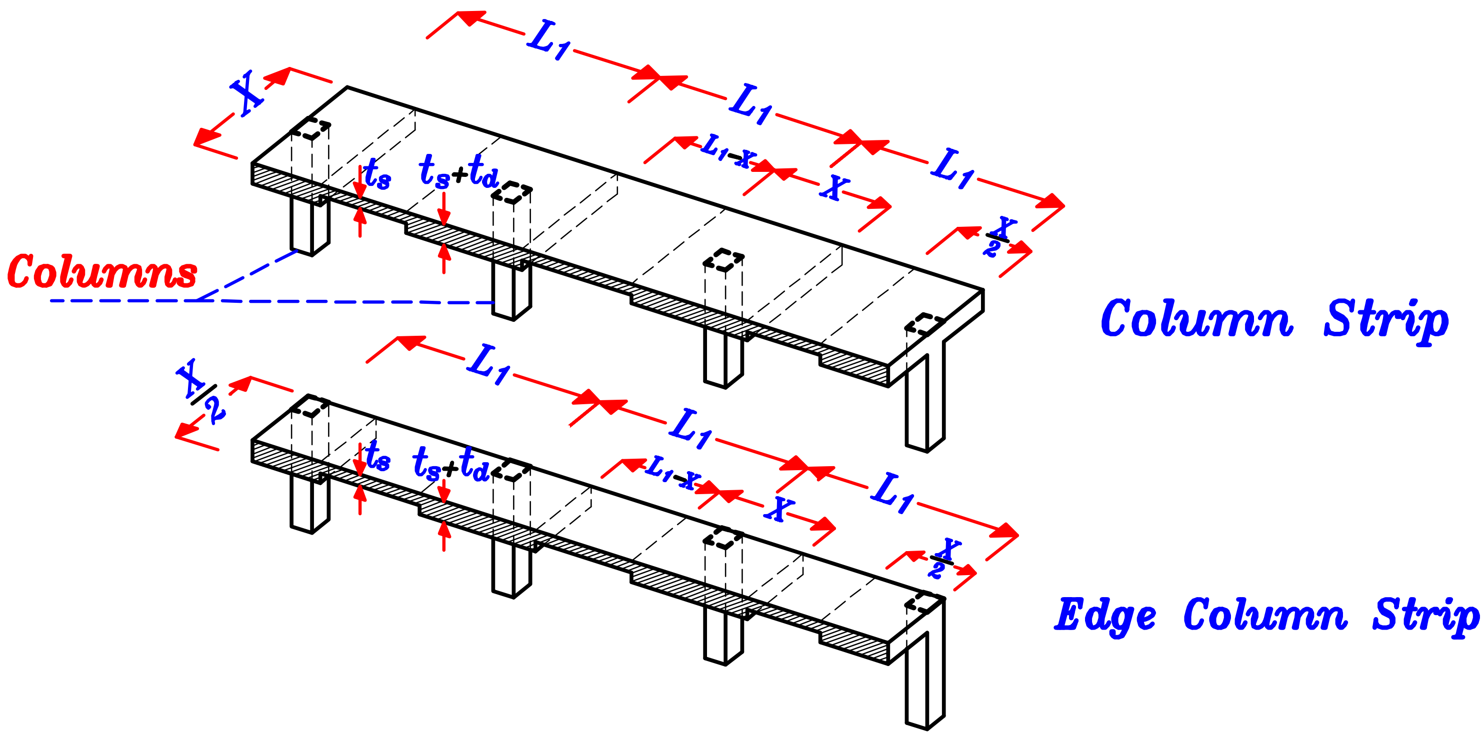


With Drop Panel. (Strips in Long Direction)

نفرض ان عرض ال Column Strip = X = عرض الdrop panel (فى الاتجاهين)

عرض ال Field Strip = باقى المسافه من C.L.to C.L.





With Drop Panel. (Strips in Long Direction)

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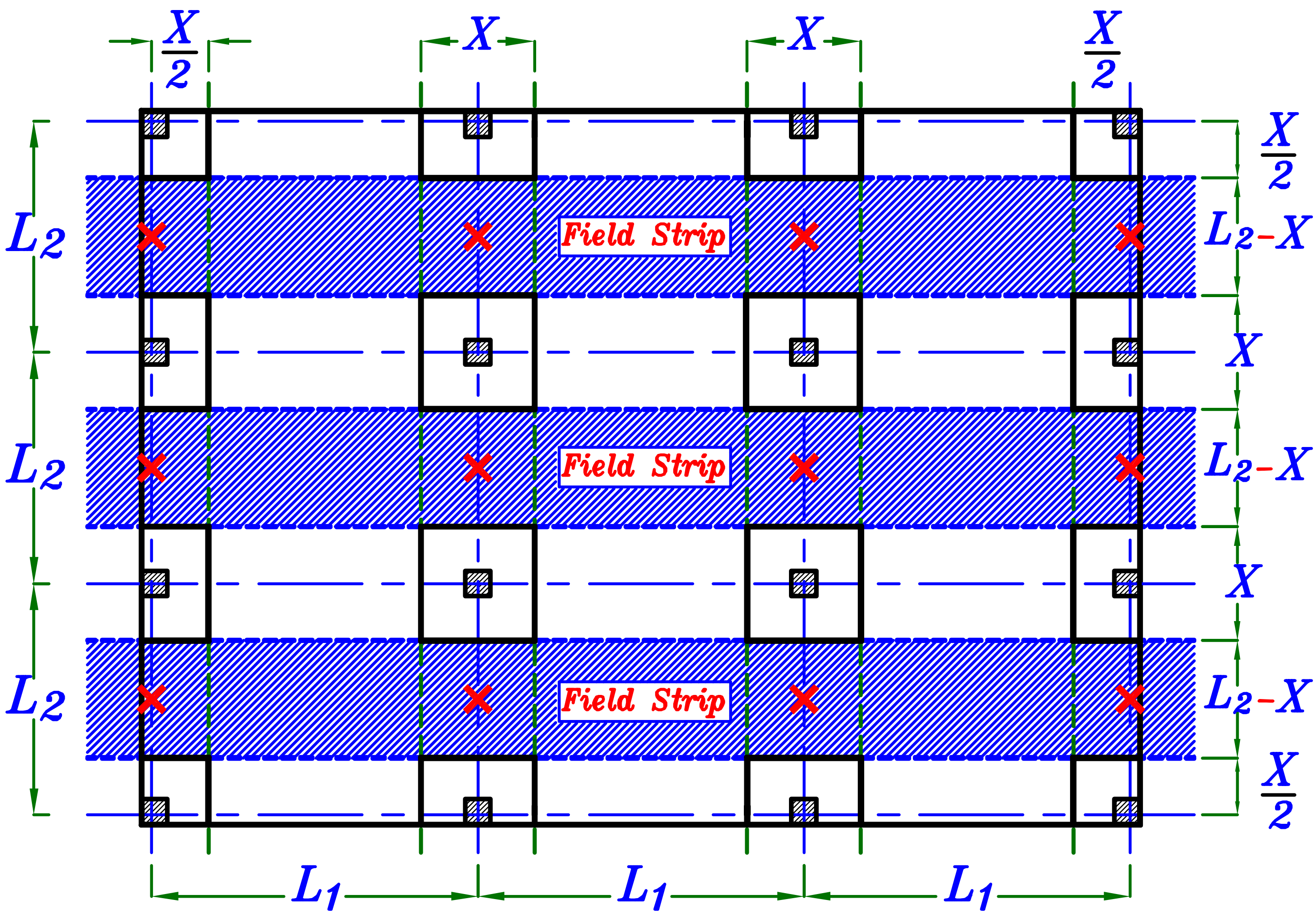
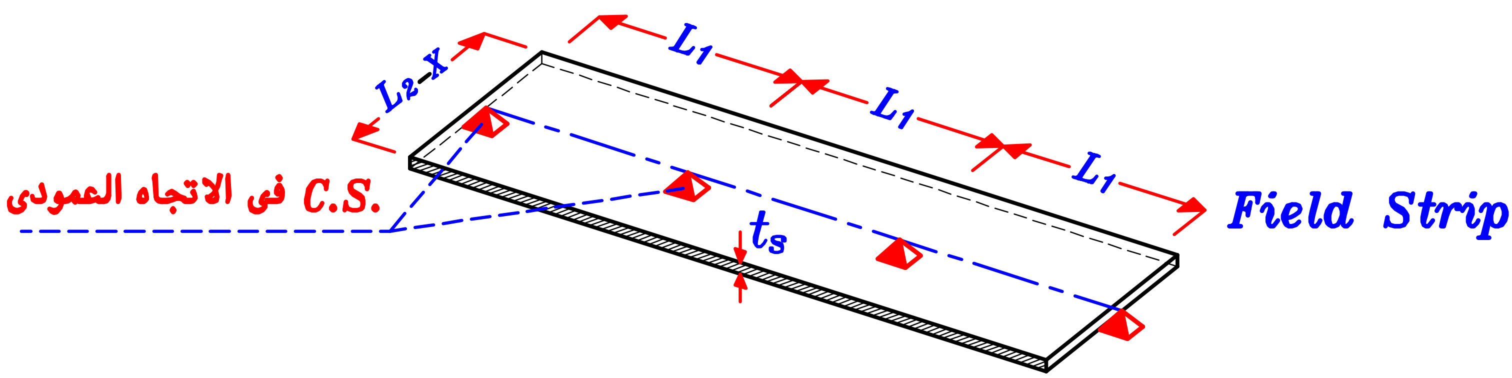




Steps of Design. خطوات تصميم ال Flat Slabs

1- Get concrete Dimensions For the slab elements.

a- Columns Dimensions. ($b_{col.}$)

b- Slab thickness (t_s) .

c- Drop Panel Dimensions. ان وجدت

2- Calculate loads on the slab (w_s) .

3- Check punching.

4- Take a Strips in the slabs at long and short directions.
strip width is From C.L. the slab to C.L. the slab.

& Draw B.M.D. For the Strip (M_o) Using

a- Empirical Method. ------------------- لو شروطها متحققة

b- Frame Analysis Method. --- لو شروط ال Empirical غير متحققة

5- Distribute the moment on both
Column Strip & Field Strip

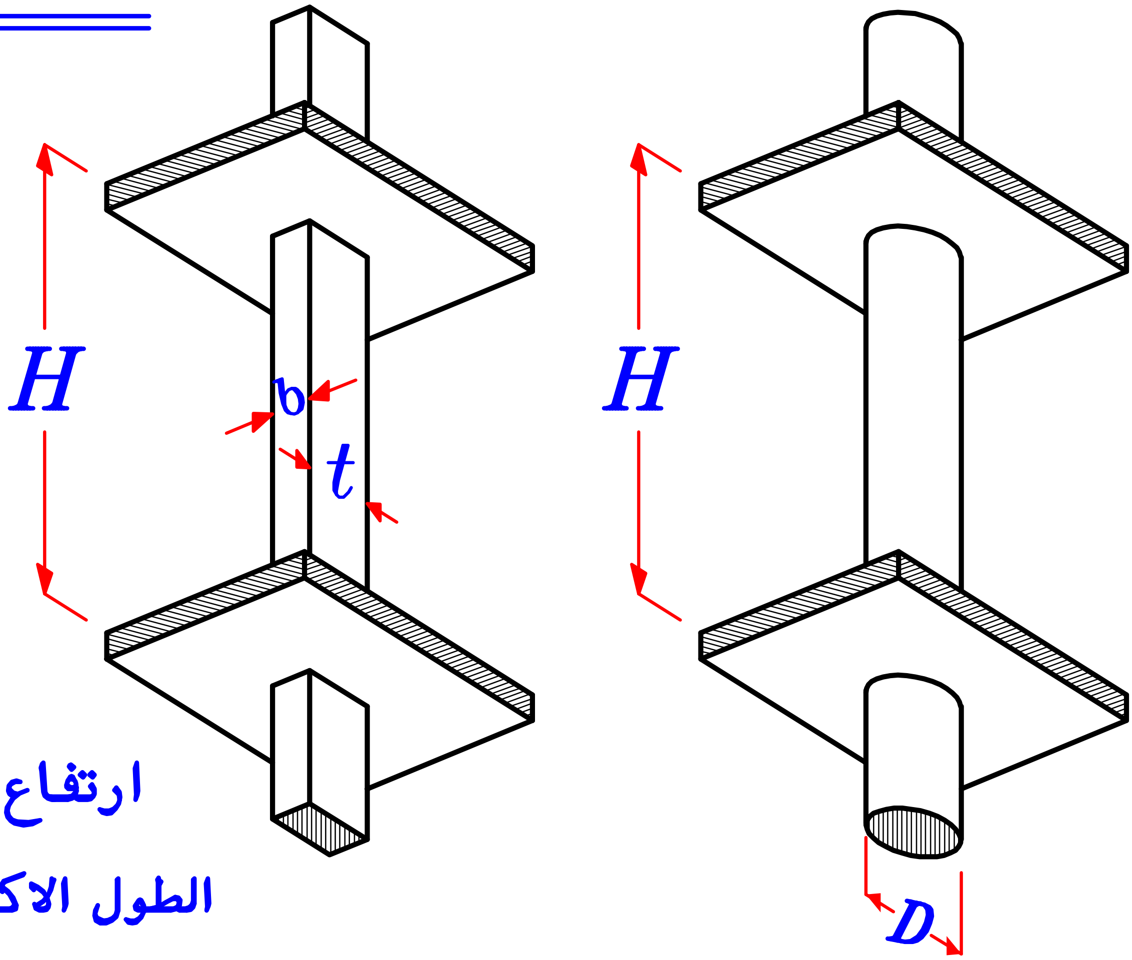
> يجب أن يكون عرض ال Column Strip فى الاتجاهين متساوى = $\frac{L_2}{2}$

6- Design the sections of the slab using Charts.

7- Draw Details of RFT. of the slab in plan.

1- Get concrete Dimensions For the Flat Slab Elements.

@ Columns.



$H =$ ارتفاع الدور

$L_1 =$ الطول الاكبر للباكيه

$b_{min.}$ (للأعمده المستطيله)

$D_{min.}$ (للأعمده الدائريه)

$$= \begin{cases} 300\ mm \\ \frac{H}{15} \\ \frac{L_1}{20} \end{cases}$$

نأخذ القيمه الأكبر

و تقرب لأقرب ٥٠ مم بالزياده

حتى يمكن للبلاطه مقاومه الـ *punching* الناتج عن العمود .

$$\boxed{b \nless 300\ mm}$$

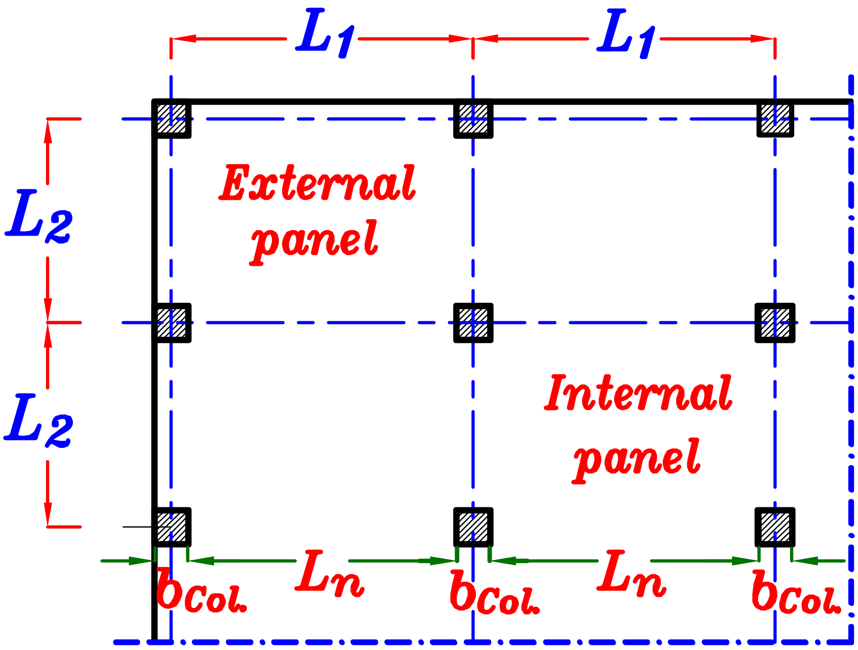
حتى يكون العمود *safe buckling* .

$$\boxed{b \nless \frac{H}{15}}$$

حتى يمكن للعمود تحمل وزن البلاطه و العزوم المتولده عليه .

$$\boxed{b \nless \frac{L_1}{20}}$$

ⓑ Thickness of the Slab. (t_s)



L_1 = The Long span of the Panel. الطول الاكبر للباكيه .

L_2 = The Short span of the Panel. الطول الاصغر للباكيه .

L_n = The longer clear span of the panel. الطول الخالص الاكبر .

أى المسافه من وجه العمود الى وجه العمود الذى يليه فى اتجاه الطول الطويل .

يعتمد إختيار تخانه البلاطه (t_s) على وجود Drop Panel أو عدم وجودها .

t_s	Slab without Drop Panel		Slab with Drop Panel	
External Panel	$t_s = \frac{L_1}{32}$	نأخذ القيمة الأكبر	$t_s = \frac{L_1}{36}$	نأخذ القيمة الأكبر
Internal Panel	$t_s = \frac{L_1}{36}$		$t_s = \frac{L_1}{40}$	

يتم تقريب (t_s) لأقرب ٢٠ مم بالزياده أو أى رقم يقبل القسمه على ٥٠ مم.

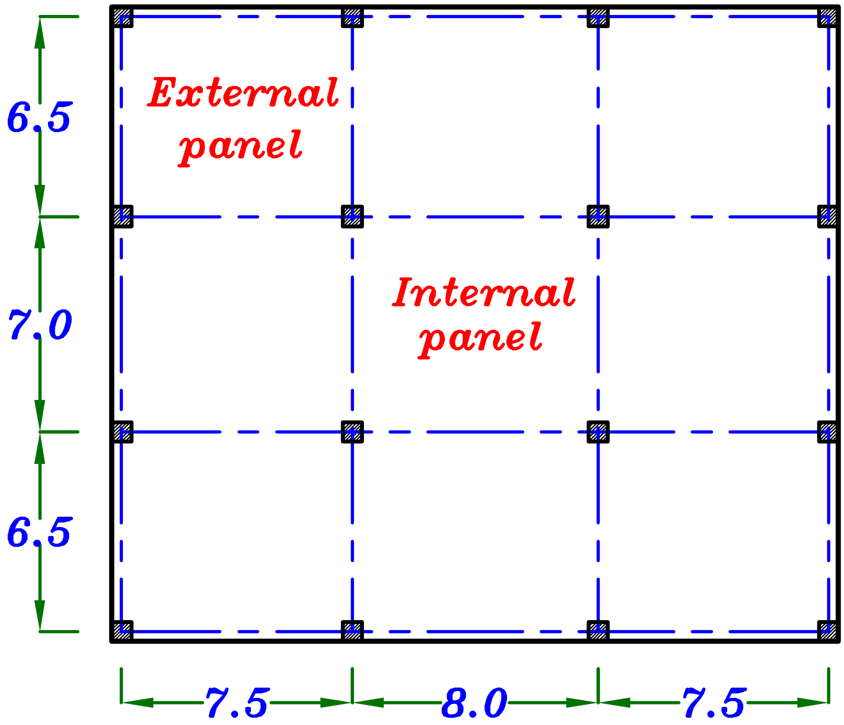
$$(t_s)_{min.} = 150\ mm$$

$t_s = 150, 160, 180, 200, 220, 240, 250, 260, 280$ ....

Example.

without drop panel

$\boxed{\text{Calculate } t_s}$



For External panel. $L_1 = 7.50$ m

For Internal panel. $L_1 = 8.0$ m

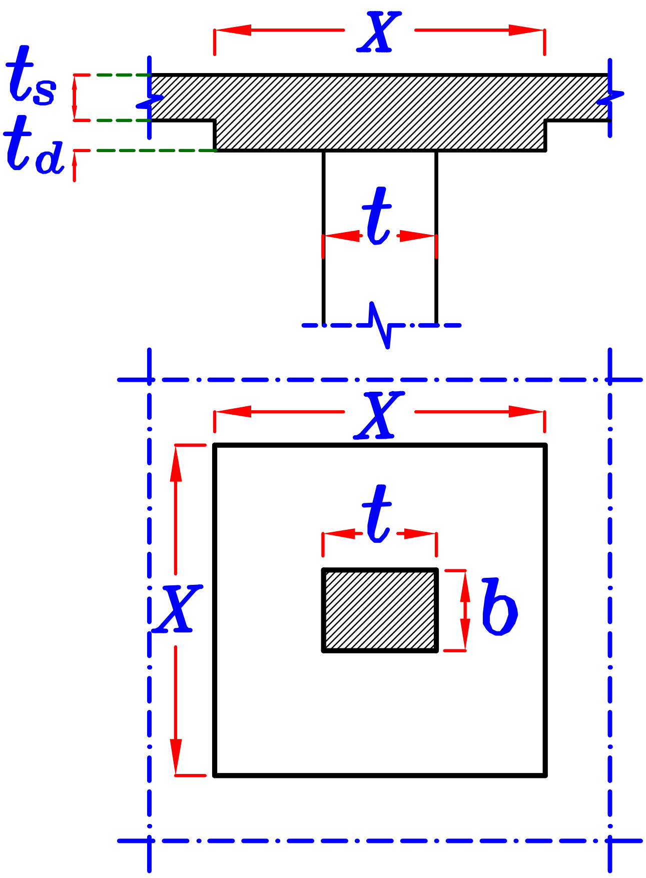
External panel $t_s = \frac{L_1}{32} = \frac{7500}{32} = 234.3$ mm

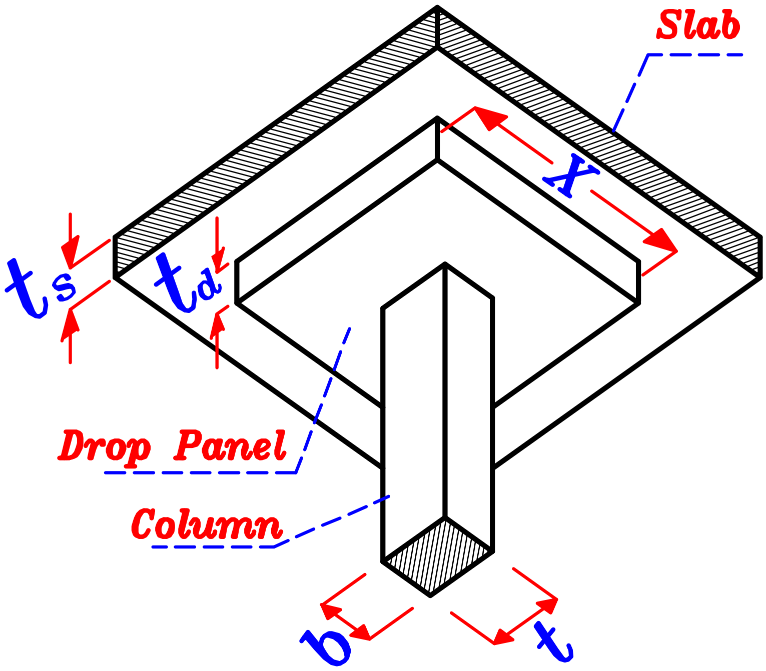
Internal panel $t_s = \frac{L_1}{36} = \frac{8000}{36} = 222.2$ mm

$t_s = 240$ mm

$\boxed{t_s = 240 \text{ mm}}$

(C) Drop Panel Dimensions. ان وجدت





حالات استخدام ال Drop Panel

* عندما يكون ال Moment $(-Ve)$ كبير .
* تستعمل فى حاله وجود أحمال حيه عاليه . $(5.0 \rightarrow 8.0)\ kN/m^2$
* فى حاله t_s كبيره .
* فى حاله البحور الكبيره .

Dimensions of the Drop Panel.

$$\frac{L_1 \text{ or } L_2}{3} \leqslant X \leqslant \frac{L_2}{2}$$

تكون غالباً مربعه $(X * X)$

ملحوظه اذا وجد فى البلاطه اكثر من قيمه ل L_2 $\quad \frac{L_2\ \text{اكبر}}{3} \leqslant X \leqslant \frac{L_2\ \text{اصغر}}{3}$

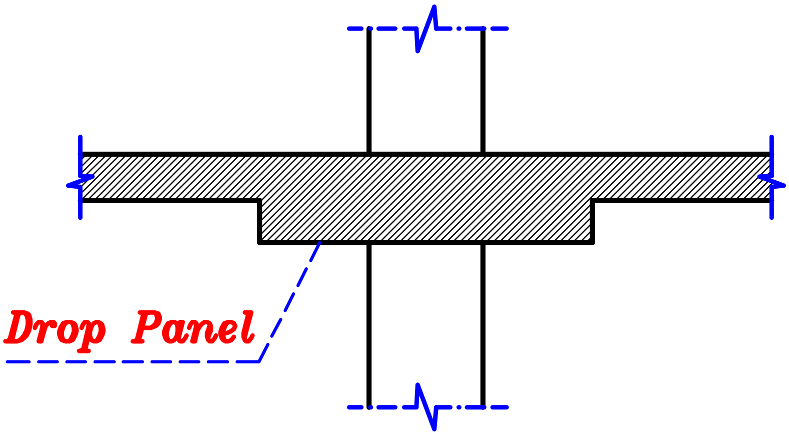
$$\frac{t_s}{4} \leqslant t_d \leqslant \frac{t_s}{2}$$

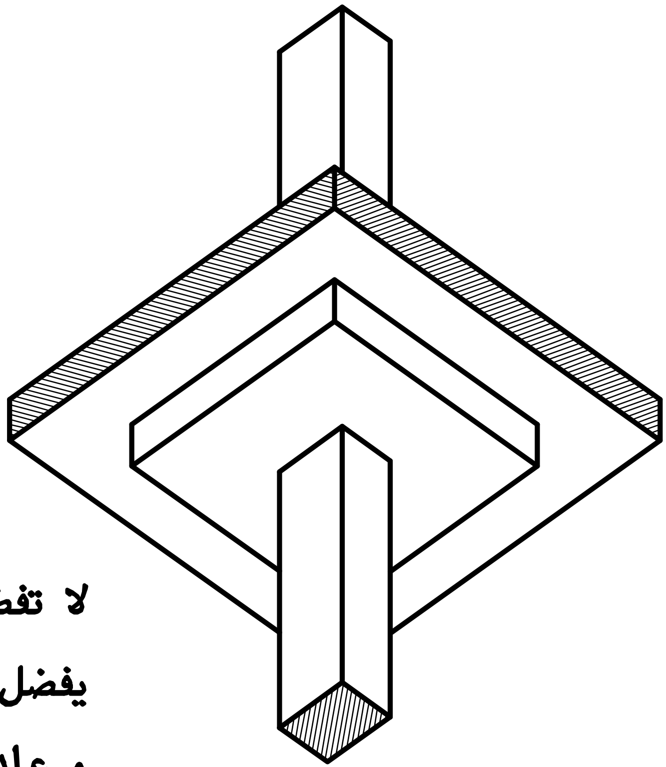
(t_d) تخانه ال Drop Panel

Take $\boxed{t_d = \frac{t_s}{2}}$ & $\boxed{X = \frac{L_2\ \text{اصغر}}{2}}$ فى الإتجاهين

تقرب لاقرب أقل رقم زوجى بالاقل

Ordinary drop panel.



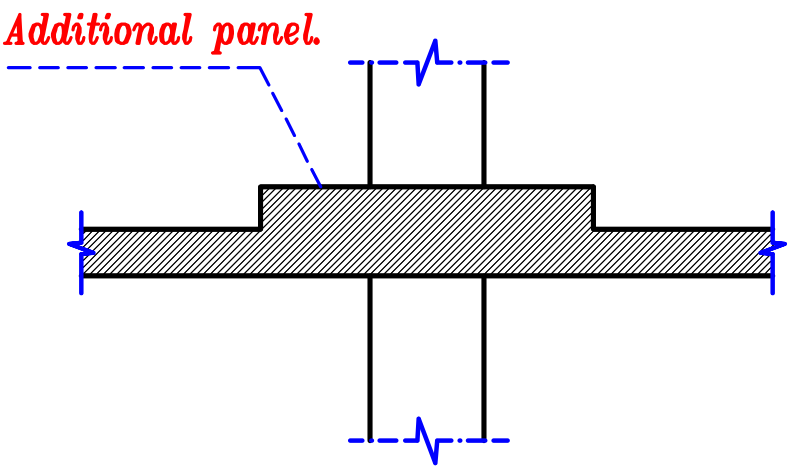


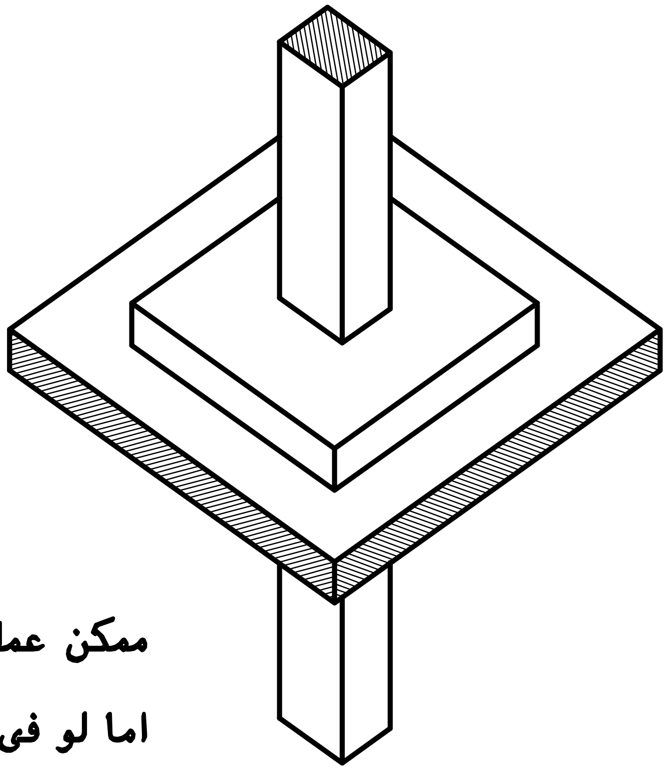
لا تفضل فى البيوت السكنيه

يفضل تغطيتها بسقف معلق (False ceiling)

و عاده يستخدم فى المكاتب و المبانى الاداريه .

Additional panel. above the slab.





ممكن عملها فى الاسطح الاخيره

اما لو فى الادوار المتكرره فيجب تغطيتها بـ Floor cover

و أبعادها نفس أبعاد الـ Drop Panel

2- Calculate loads on the slab (W_s).

ⓐ Without Drop Panel.

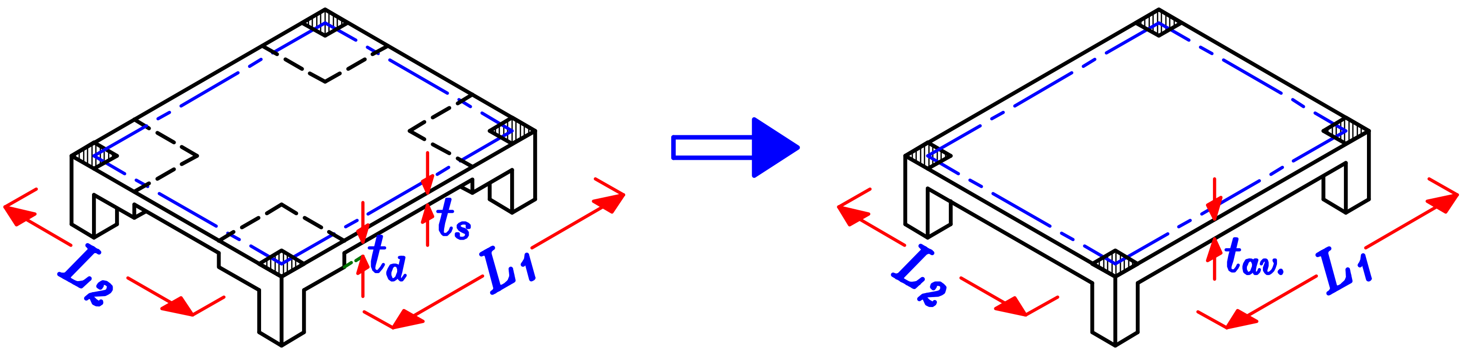
$$\boxed{(W_s)_{U.L.} = 1.4\left[t_s \gamma_c + F.C. + Wall\right] + 1.6(L.L.)}$$

ⓑ With Drop Panel.

$$\boxed{(W_s)_{U.L.} = 1.4\left[(t_{s\,av.})\gamma_c + F.C. + Wall\right] + 1.6(L.L.)}$$

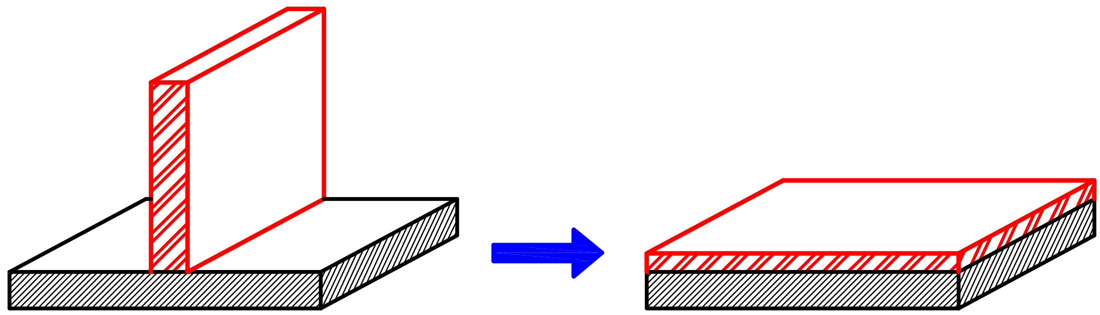
where: $\boxed{t_{s\,av.} = \left(t_s + \frac{t_d}{4}\right)}$

كأننا وضعنا حمل ال **drop panel**
كحمل منتظم على كل الباكيه .



- **Weight of Walls** ($kN \backslash m^2$)

نعمل على توزيع وزن الحائط المركز الى حمل موزع على البلاطه .



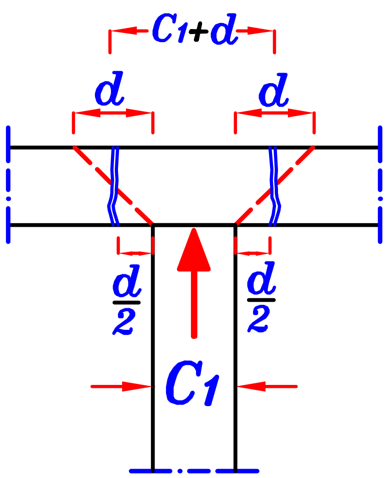
Take Walls (working) = $(1.5 \rightarrow 2.0)$ $(kN \backslash m^2)$

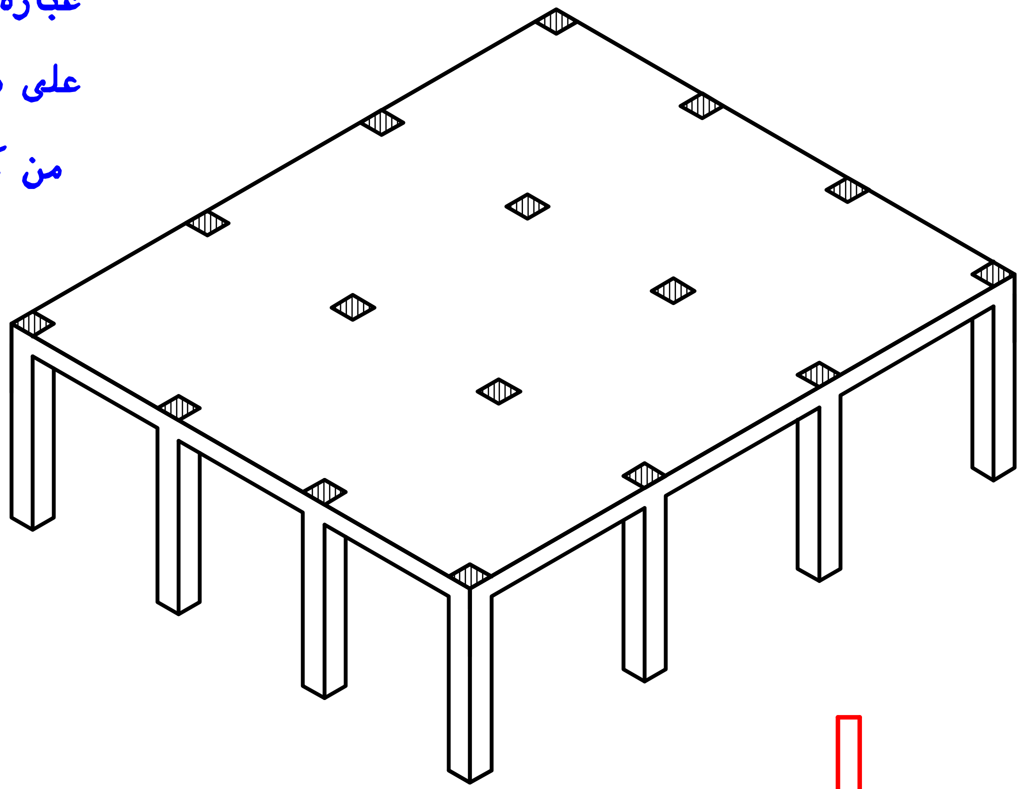
> **ملحوظه** في حاله ال **Solid Slab** لا يتم حساب وزن الحوائط مع ال W_s
> لان الحوائط تكون محموله مباشره على الكمرات .

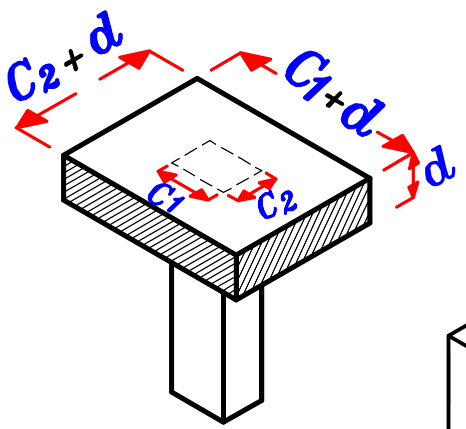
3- Check Punching shear. القص الثاقب

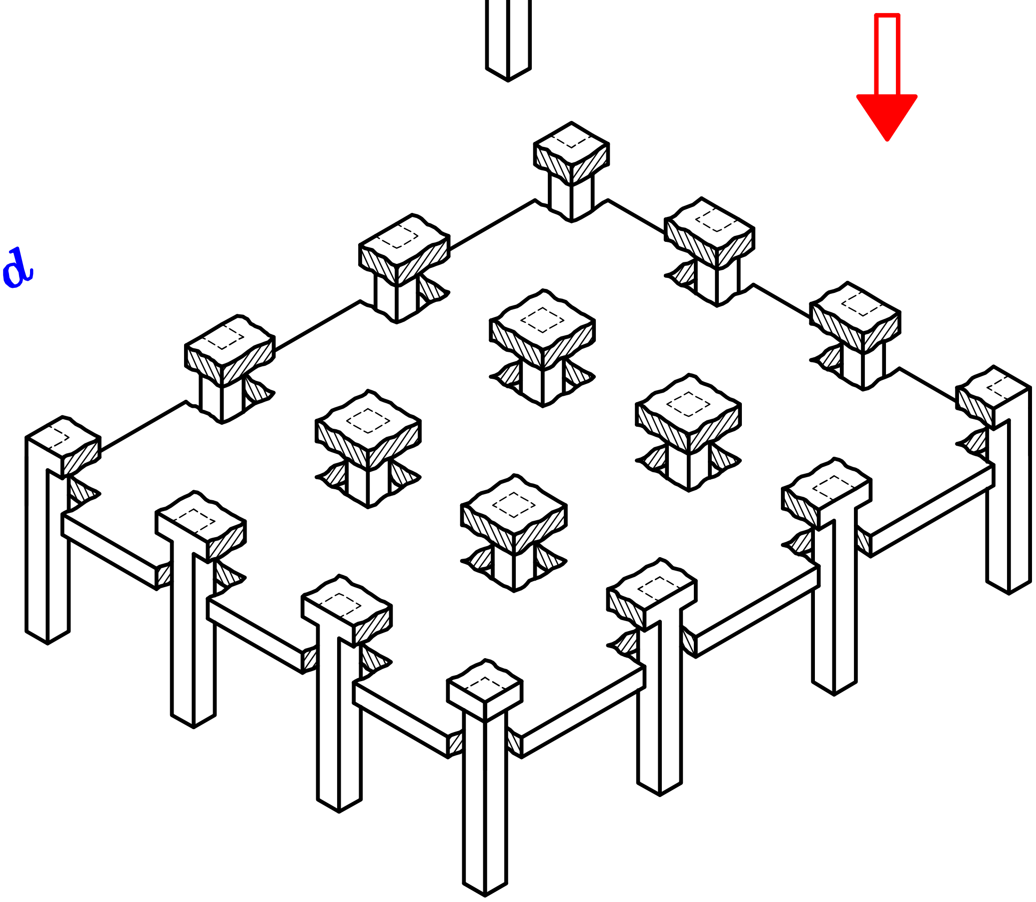
القطاع الحرج فى القص الثاقب عباره عن المحيط الذى يحيط بالعمود على مسافه $\frac{d}{2}$ من وجه العمود من كل جهه

يجب التأكد من أن العمود لن يخترق البلاطه.



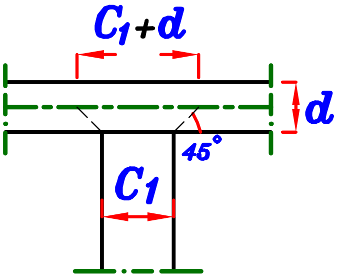






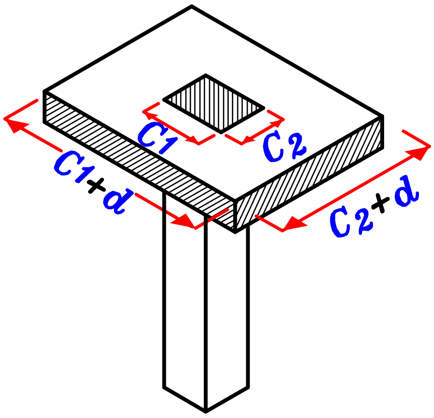
و للتأكد من ذلك نحسب q_{pu} و هو اجهاد القص الذى سينتج عن ثقب العمود للبلاطه.

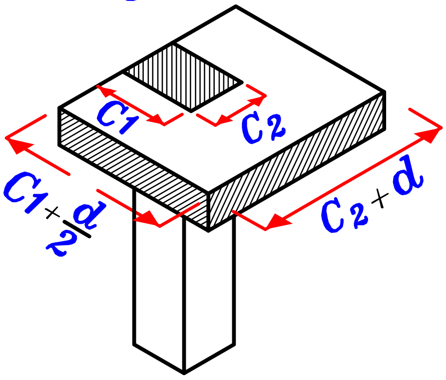
و نحسب q_{pcu} و هى مقاومه الخرسانه للقص الناتج عن ثقب البلاطه.

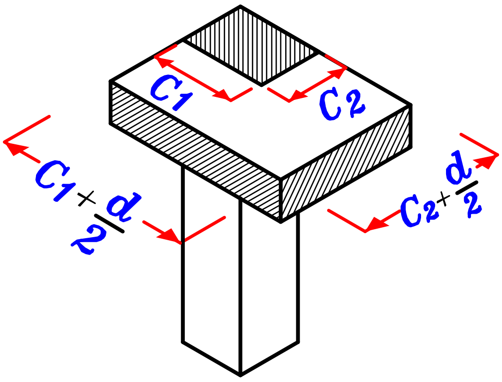


b_o هو محيط الخرسانه التى سيحدث لها *punching*

Interior Col.	Edge Col.	Corner Col.







$$b_o = 2(C_1+d) + 2(C_2+d)$$

$$b_o = 2\left(C_1+\frac{d}{2}\right) + (C_2+d)$$

$$b_o = \left(C_1+\frac{d}{2}\right) + \left(C_2+\frac{d}{2}\right)$$

لحساب قيمه q_{pcu} و هى مقاومه الخرسانه للقص الناتج عن ثقب البلاطه.

نأخذ القيمه الاقل من الاربع قيم التاليه .

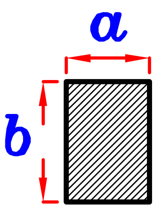
$$\boxed{q_{pcu} = 0.8\left(\frac{\alpha\, d}{b_o} + 0.2\right)\sqrt{\frac{F_{cu}}{\gamma_c}}}$$

$\alpha = 4$ Interior Col.

$\alpha = 3$ Edge Col.

$\alpha = 2$ Corner Col.

$$\boxed{q_{pcu} = 0.316\left(0.5 + \frac{a}{b}\right)\sqrt{\frac{F_{cu}}{\gamma_c}}} \quad (N/mm^2)$$



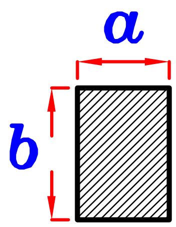
a هو العرض الصغير

$$\boxed{q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}}} \quad (N/mm^2)$$

$$\boxed{q_{pcu} = 1.60} \quad (N/mm^2)$$

في قانون

$$q_{pcu} = 0.316 \left(0.5 + \frac{a}{b}\right) \sqrt{\frac{F_{cu}}{\gamma_c}}$$

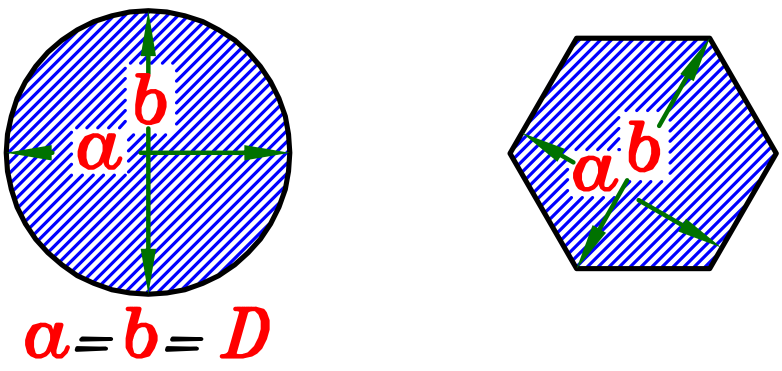


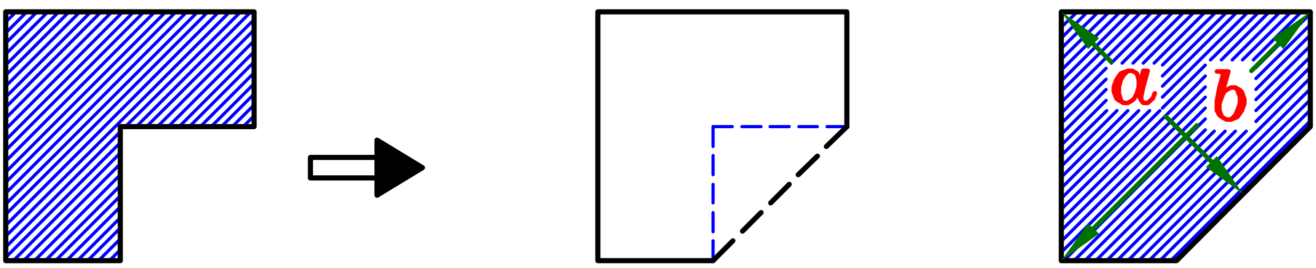
a هو العرض الاصغر للعمود المستطيل

b هو العرض الاكبر للعمود المستطيل

اذا كان مسطح التحميل شكل غير المستطيل

يوخذ شكل مسطح التحميل اقل ما يمكن و تكون قيمه b هى اطول بُعد فى مسطح التحميل الفعال و قيمه a هى اطول بُعد عمودى على b فى مسطح التحميل .





شكل العمود الاصلى

شكل سطح التحميل الفعال

و عاده تكون القيمه الاقل من الاربع قوانين لل q_{pcu} هى قيمه

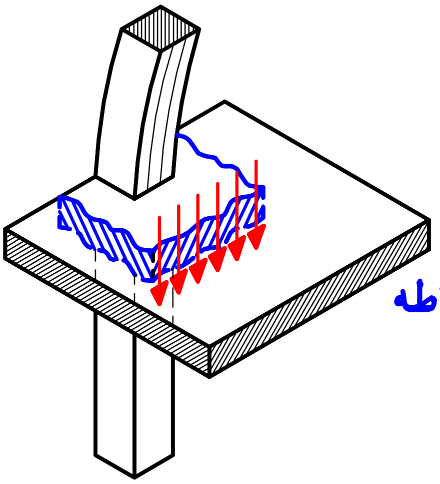
$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} \quad (N/mm^2)$$

q_{pu} = Shear Force acting around the column.

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta$$

Q_{pu} الحمل الكلى الذى يحمله العمود فى الدور الواحد مطروحا منه الحمل الواقع مباشره فوق قطاع العمود

A_p المساحه المقاومه لل punching



يتم ضرب اجهاد القص الثاقب المؤثر فى معامل β لتعويض ال punching الغير محسوب و الناتج عن عزوم الاعمده .

كلما زاد ال moment على العمود كلما زاد ال punching على البلاطه

و تتوقف قيمه β على مكان العمود .

β:

- 1.15 (Interior Column). العزم على العمود قليل لذا قيمه β صغيره
- 1.30 (Edge Column). العزم متوسط على العمود لذا قيمه β متوسطه
- 1.50 (Corner Column). العزم كبير على العمود لذا قيمه β كبيره

---

* IF $q_{pu} \leqslant q_{p\,cu}$ $\longrightarrow$ Safe punching shear.
* IF $q_{pu} > q_{p\,cu}$ $\longrightarrow$ UnSafe punching shear.

اذا كانت البلاطه unsafe punching فيجب عمل احدى الاتى :

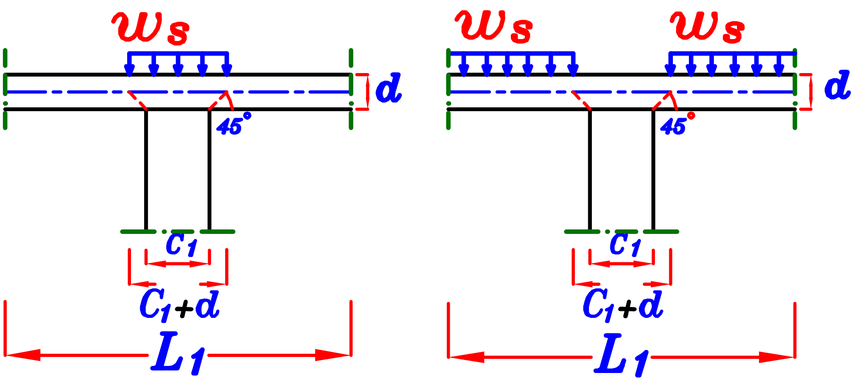
عمل drop panel أو Column head أو زياده قيمه t_s أو زياده أبعاد العمود

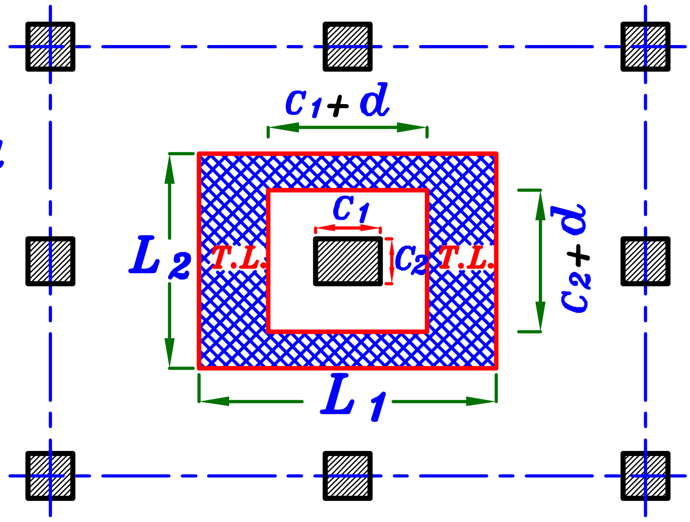
فى الدراسه فى الكليه يتم عمل check punching لعمود واحد فقط

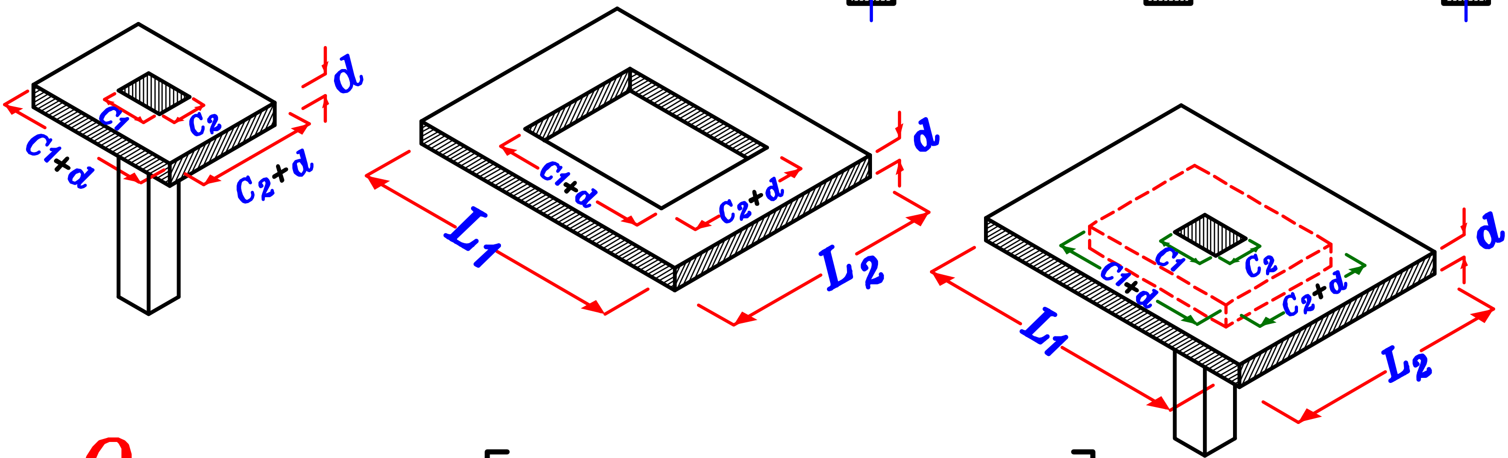
و يفضل ان يكون عمود داخلى Interior Column

أما فى العمل فيجب عمل check punching على كل الاعمده .

1- Interior Column. عمود داخلى







$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1+d)(C_2+d) \right]$$

$$A_p = (b_o * d) = \left[2(C_1+d) + 2(C_2+d) \right] * d$$

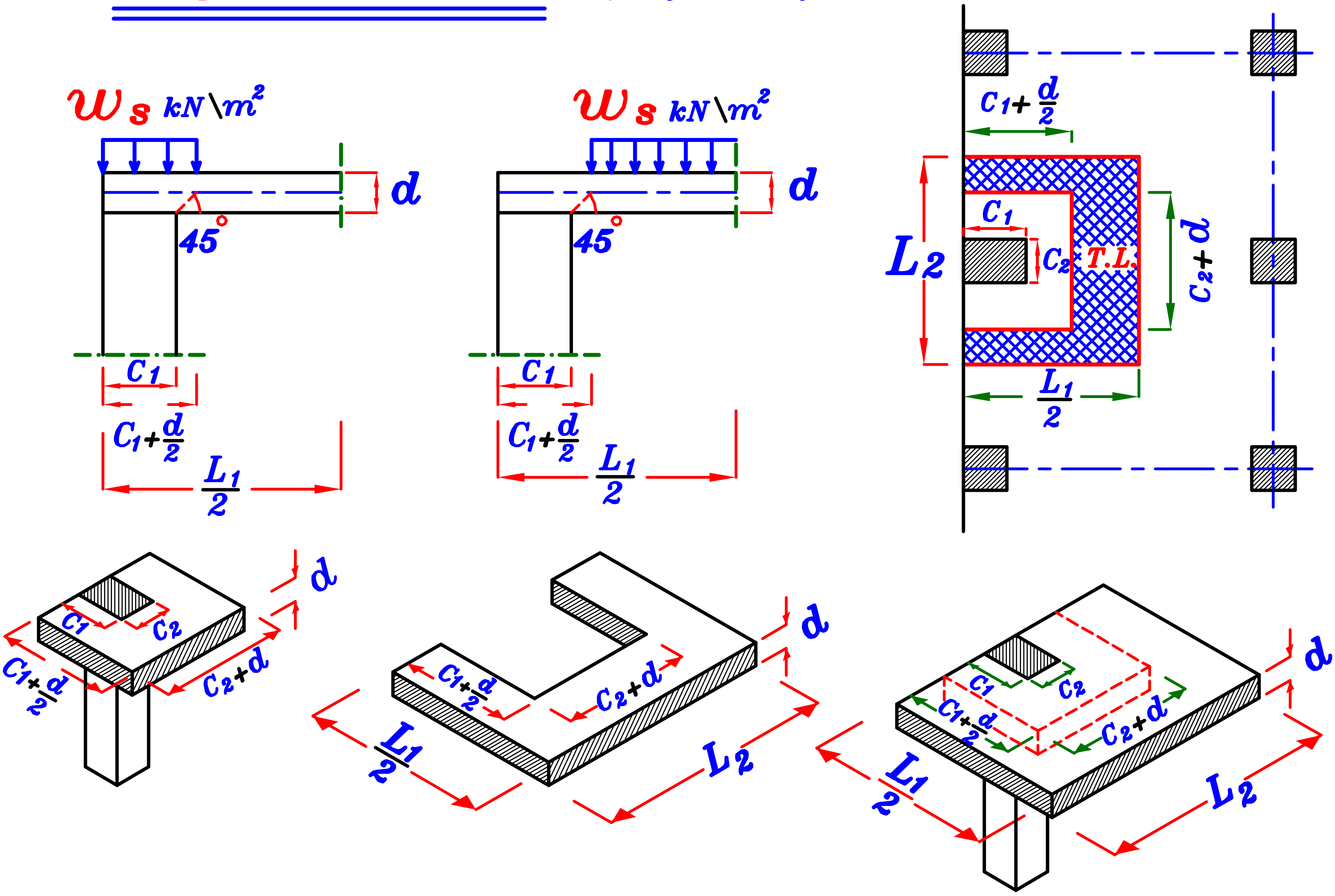
$$- \; q_{pu} = \frac{Load}{Area} = \frac{Q_{pu}}{A_p} * \beta$$

$$\boxed{\beta = 1.15 \; (\text{For Interior Column})}$$

$$- \; q_{pcu} = \text{as before} \simeq 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}}$$

* IF $q_{pu} \leqslant q_{pcu} \longrightarrow$ Safe punching shear.
* IF $q_{pu} > q_{pcu} \longrightarrow$ UnSafe punching shear.

2-Edge Column. عمود طرفى



$$Q_{pu} = W_s\left[\frac{L_1}{2} * L_2 - \left(C_1+\frac{d}{2}\right)(C_2+d)\right]$$

$$A_p = (b_o * d) = \left[2\left(C_1+\frac{d}{2}\right)+(C_2+d)\right] * d$$

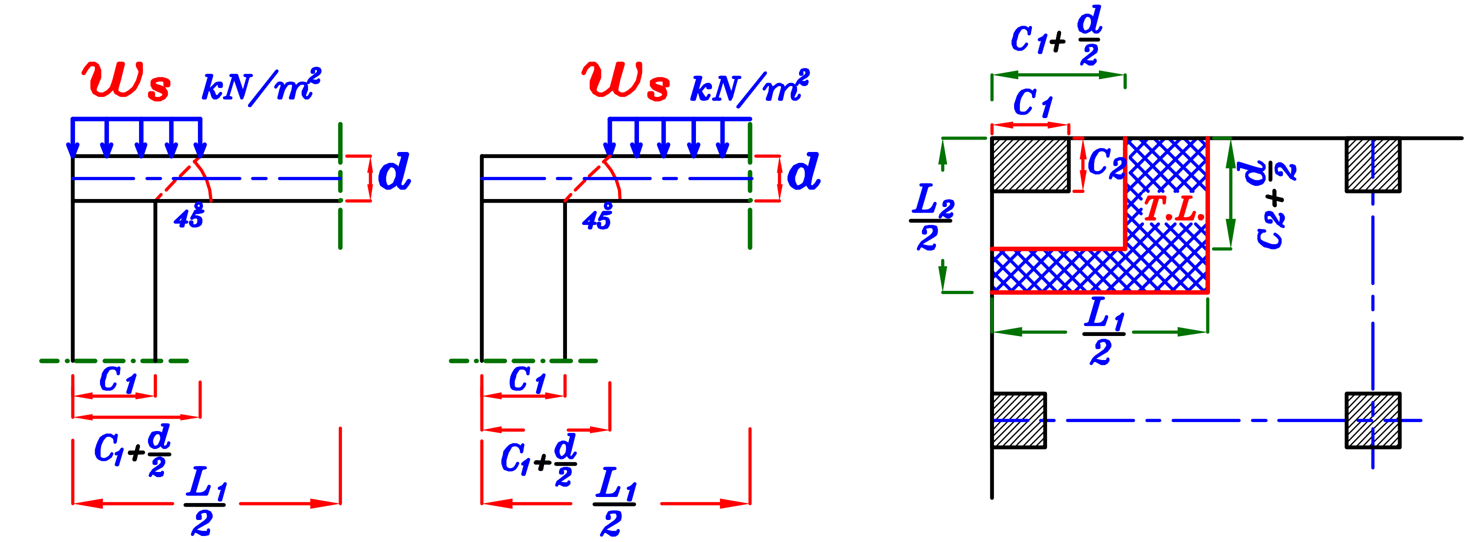
$$- \; q_{pu} = \frac{Load}{Area} = \frac{Q_{pu}}{A_p} * \beta$$

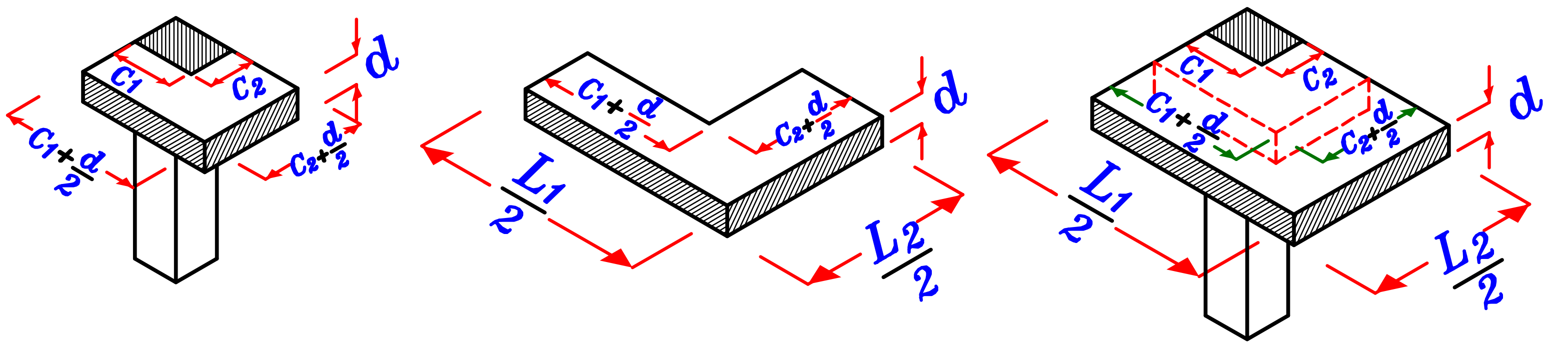
$$\boxed{\beta = 1.30 \quad (\text{For Edge Column})}$$

$$- \; q_{pcu} = \text{as before} \simeq 0.316\sqrt{\frac{F_{cu}}{\gamma_c}}$$

* IF $q_{pu} \leq q_{pcu} \longrightarrow$ Safe punching shear.
* IF $q_{pu} > q_{pcu} \longrightarrow$ UnSafe punching shear.

3- Corner Column. عمود ركنى





$$Q_{pu} = W_s \left[\frac{L_1}{2} * \frac{L_2}{2} - \left(C_1+\frac{d}{2}\right)\left(C_2+\frac{d}{2}\right)\right]$$

$$A_p = (b_o * d) = \left[\left(C_1+\frac{d}{2}\right)+\left(C_2+\frac{d}{2}\right)\right] * d$$

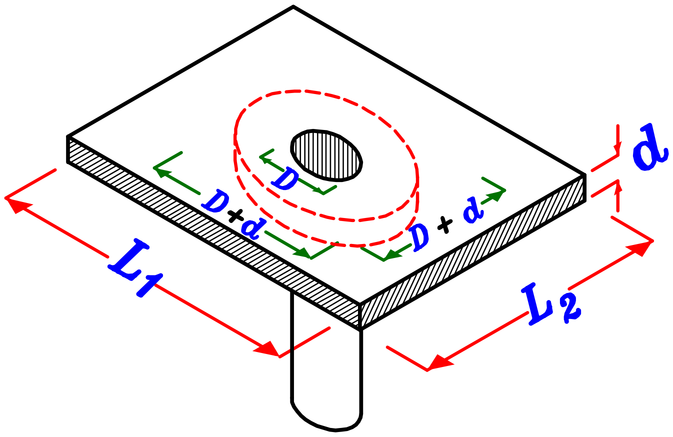
$$- \; q_{pu} = \frac{Load}{Area} = \frac{Q_{pu}}{A_p} * \beta$$

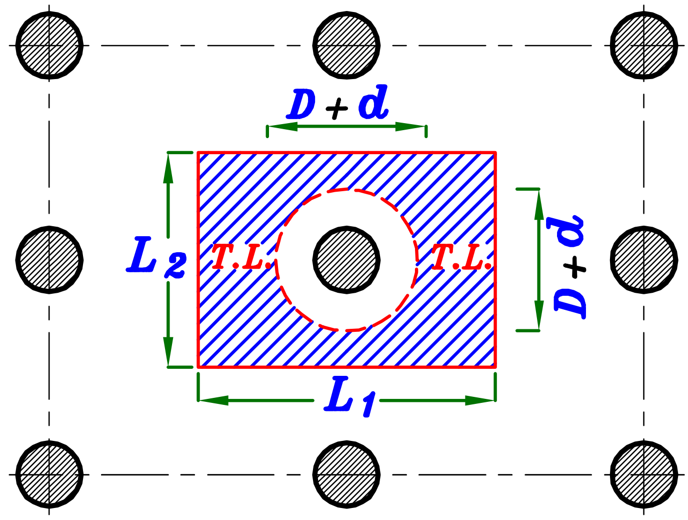
$$\boxed{\beta = 1.50 \quad (\text{For Corner Column})}$$

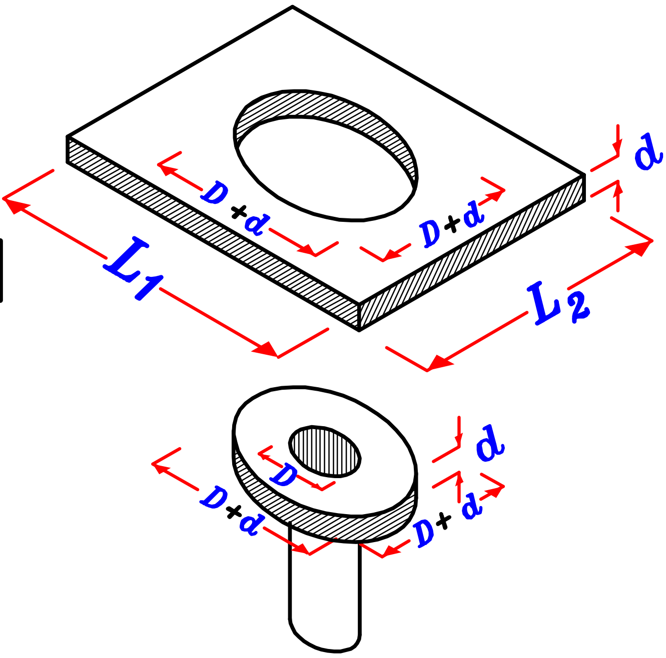
$$- \; q_{pcu} = \text{as before} \simeq 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}}$$

* IF $q_{pu} \leqslant q_{pcu} \longrightarrow$ Safe punching shear.
* IF $q_{pu} > q_{pcu} \longrightarrow$ UnSafe punching shear.

Circular Column. العمود الدائرى







$$Q_{pu} = w_s \left[L_1 * L_2 - \frac{\pi\,(D+d)^2}{4}\right]$$

$$A_p = (b_o * d) = [\,\pi\,(D+d)\,] * d$$

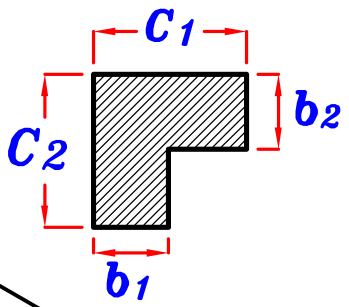
$$- \; q_{pu} = \frac{Load}{Area} = \frac{Q_{pu}}{A_p} * \beta$$

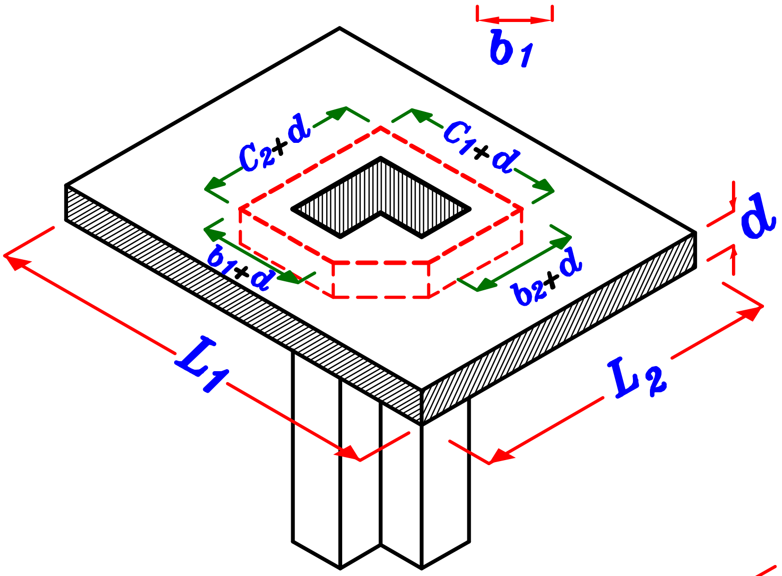
$$\boxed{\beta = 1.15 \quad (\text{For Interior Column})}$$

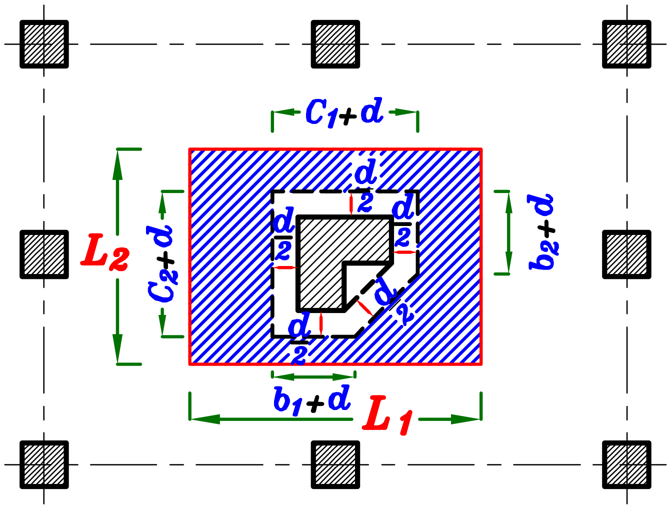
$$- \; q_{pcu} = \text{as before} \simeq 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}}$$

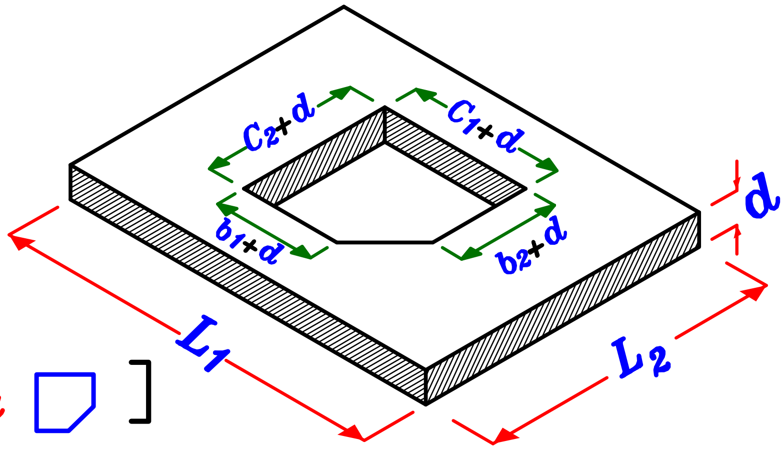
* IF $q_{pu} \leqslant q_{pcu}$ $\longrightarrow$ Safe punching shear.
* IF $q_{pu} > q_{pcu}$ $\longrightarrow$ UnSafe punching shear.

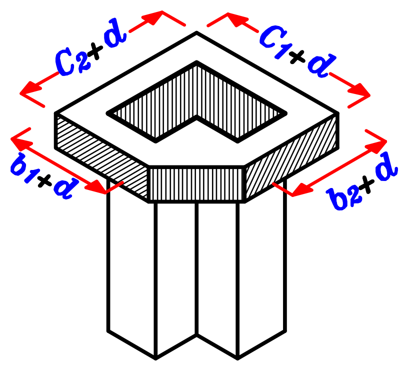
Inner L–Section Column.











$$Q_{pu} = w_s \left[L_1 * L_2 - \text{area} \right]$$

$$A_p = (b_o * d) = \left[\text{perimeter} \right] * d$$

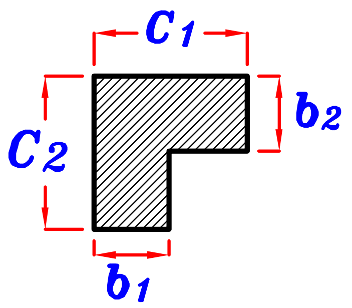
$$q_{pu} = \frac{Load}{Area} = \frac{Q_{pu}}{A_p} * \beta$$

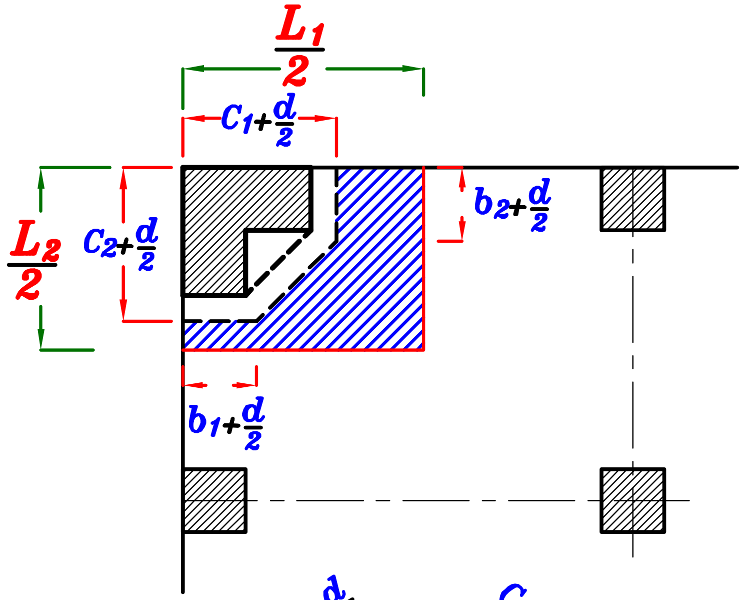
$$\boxed{\beta = 1.15 \quad (\text{For Interior Column})}$$

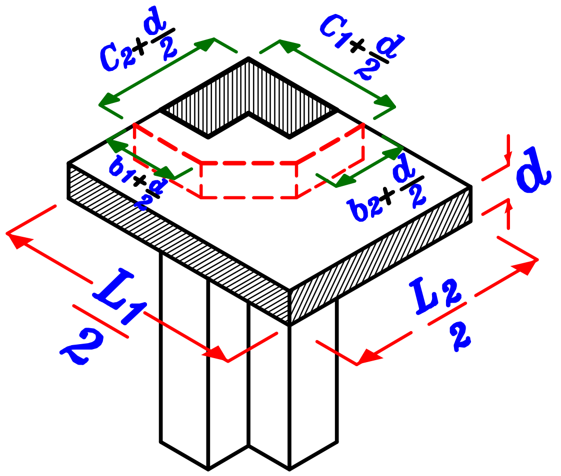
– q_{pcu} = as before $\simeq 0.316 \sqrt{\dfrac{F_{cu}}{\gamma_c}}$

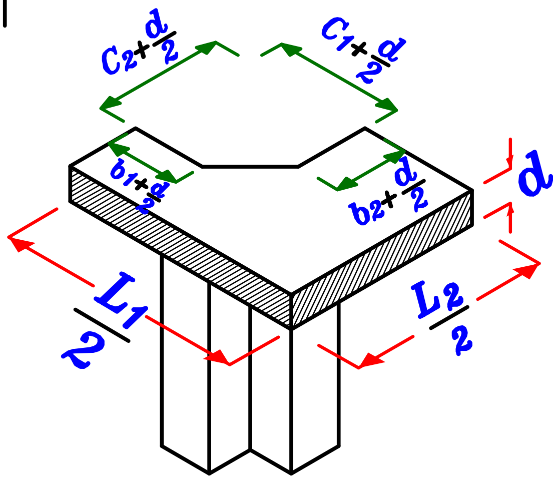
* IF $q_{pu} \leqslant q_{pcu}$ ⟶ Safe punching shear.
* IF $q_{pu} > q_{pcu}$ ⟶ Unsafe punching shear.

Corner L–Section Column.



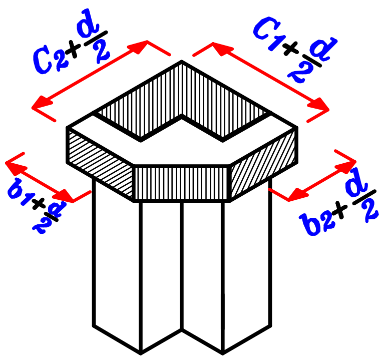






$$Q_{pu} = w_s \left[\frac{L_1}{2} * \frac{L_2}{2} - \text{area} \right]$$

$$A_p = (b_o * d) = [\text{perimeter}] * d$$



$$q_{pu} = \frac{\text{Load}}{\text{Area}} = \frac{Q_{pu}}{A_p} * \beta$$

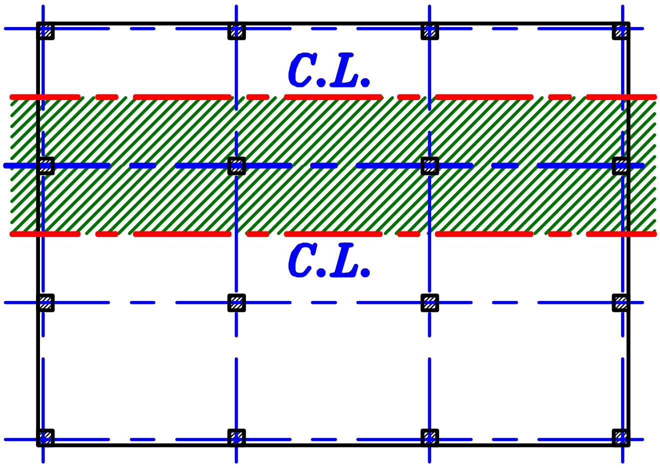
$$\beta = 1.50 \quad (\text{For Corner Column})$$

$$- \quad q_{pcu} = \text{as before} \simeq 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}}$$

* IF $q_{pu} \leqslant q_{pcu}$ ⟶ Safe punching shear.
* IF $q_{pu} > q_{pcu}$ ⟶ Unsafe punching shear.

4-Take a Strips in slabs at the long and short directions.
The strip width From C.L. the slab to C.L. the slab.

نختار محور للاعمده فى كلا من الاتجاهين (يفضل ان يكون المحور فى المنتصف)
و نأخذ شريحه فى اتجاهه يكون عرضها المسافه من C.L. البلاطه على
يمين المحور الى الـ C.L. البلاطه على يسار المحور .



ثم نرسم الـ moment على هذه الشريحه و يسمى (M_o)

باستخدام احدى الطريقتين :

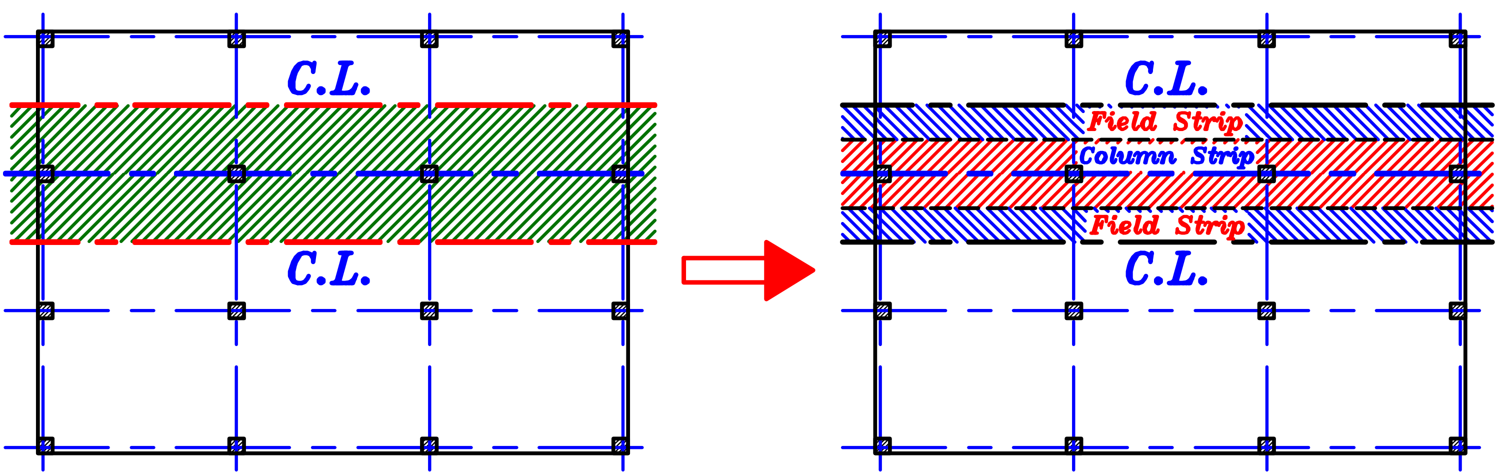
a- Empirical Method.

b - Frame Analysis Method.

ثم نوزع العزم (M_o) على جزئين و بنسب تقريبيه محفوظه

الجزء الاكبر يذهب الى الـ Column strip

و الجزء الاصغر يذهب الى الـ Field strip



و عاده نستخدم طريقه **Empirical Method** و اذا لم تنفع احدى شروطها نستخدم طريقه **Frame Analysis Method**

---

1 – Using Empirical Method.

a – Calculate the total moment on the panel (M_o)

b – Distribute (M_o) on both Column strip & Field strip.

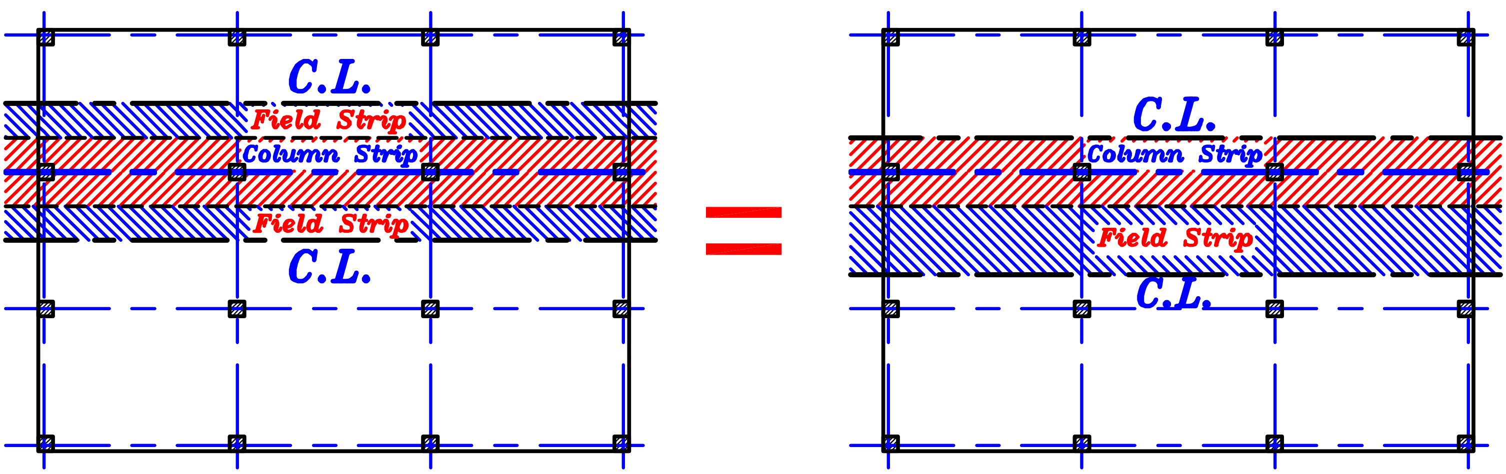
2 – Using Frame Analysis Method.

a – Calculate the total moment on the panel as a Frame (M_o)

b – Distribute (M_o) on both Column strip & Field strip.

يتم توزيع العزم على الباكيه كلها على جزئين

جزء يذهب الى شريحه العمود **Column Strip** و باقى العزم يذهب الى شريحه الوسط **Field Strip**.



① Empirical Method.

هناك عده شروط لكى نستطيع أن نستخدم الـ Empirical Method

١- لا يقل عدد البواكى عن ٣ بواكى فى اياً من الاتجاهين .

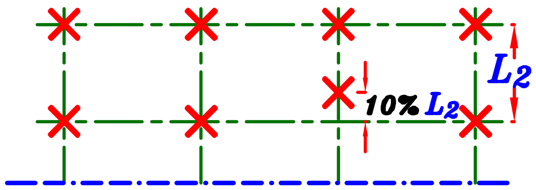
٢- أن لا تزيد نسبه طول الباكيه الواحده الى عرضها عن $\frac{٤}{٣}$ $\boxed{\frac{L_1}{L_2} \ngtr \frac{4}{3}}$

٣- الفرق فى طول أو عرض أى باكيتين متجاورتين لا يزيد عن ١٠ ٪ من البحر الاكبر .

٤- الفرق فى طول أو عرض أى باكيتين غير متجاورتين لا يزيد عن ٢٠ ٪ من البحر الاكبر .

٥- البحور الخارجيه يجب أن تكون أقل من أو تساوى البحور الداخليه .

٦- يجب أن تكون الاعمده موضوعه على خطوط مستقيمه أو بتفاوت لا يزيد عن ١٠ ٪ من طول الباكيه .



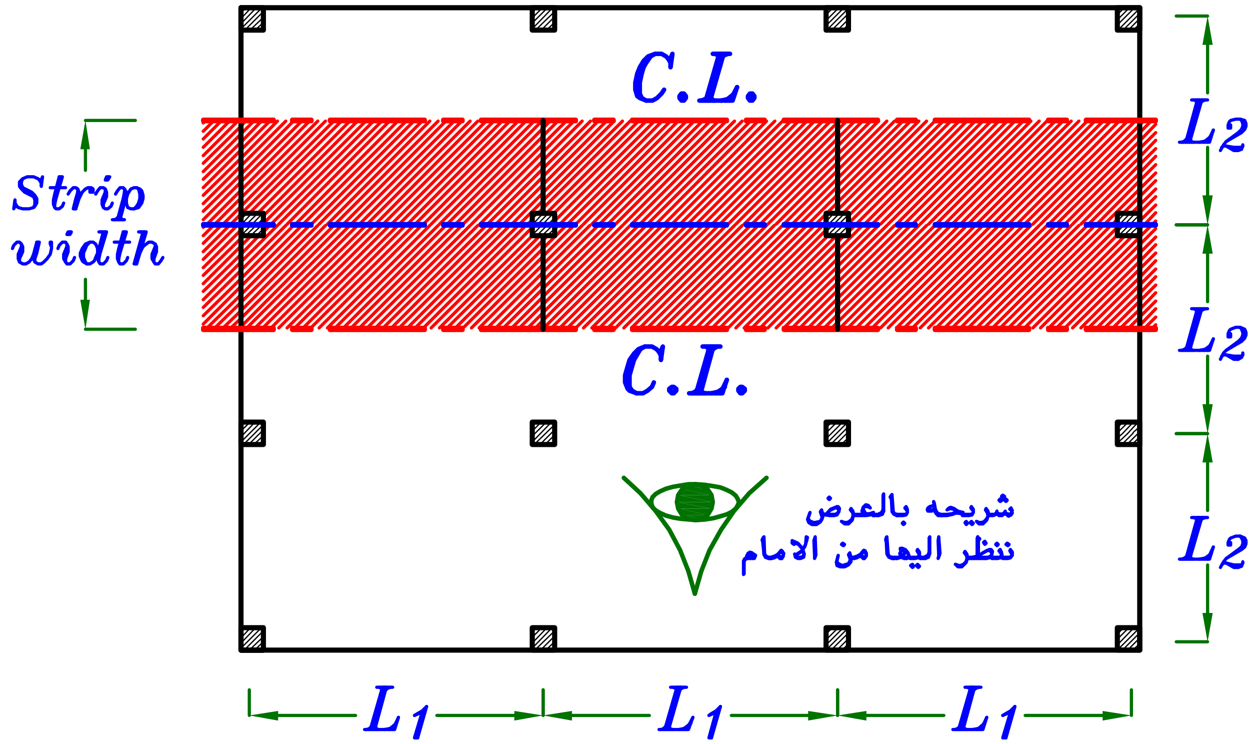
٧- فى حاله اختلاف البحور فى الباكيه الواحده نستخدم البحر الاكبر فى حساب العزم .

٨- يجب أن لا يزيد L.L. عن ضعف الـ D.L. $\boxed{L.L. \ngtr 2\left[t_s \gamma_c + F.C. + Wall\right]}$

يتم أخذ شرحتين للبلاطه شريحه فى كلا من الاتجاهين . و يكون عرض الشريحه فى كل اتجاه هو عرض الباكيه أى من C.L. to C.L. و يتم حساب empirical moment على الشريحه بالكامل ثم يتم تقسيم هذه الشريحه الى شريحتين هما :
شريحه العمود Column strip و شريحه الوسط Field strip
و بالتالى نقسم الـ empirical moment على هاتين الشريحتين .

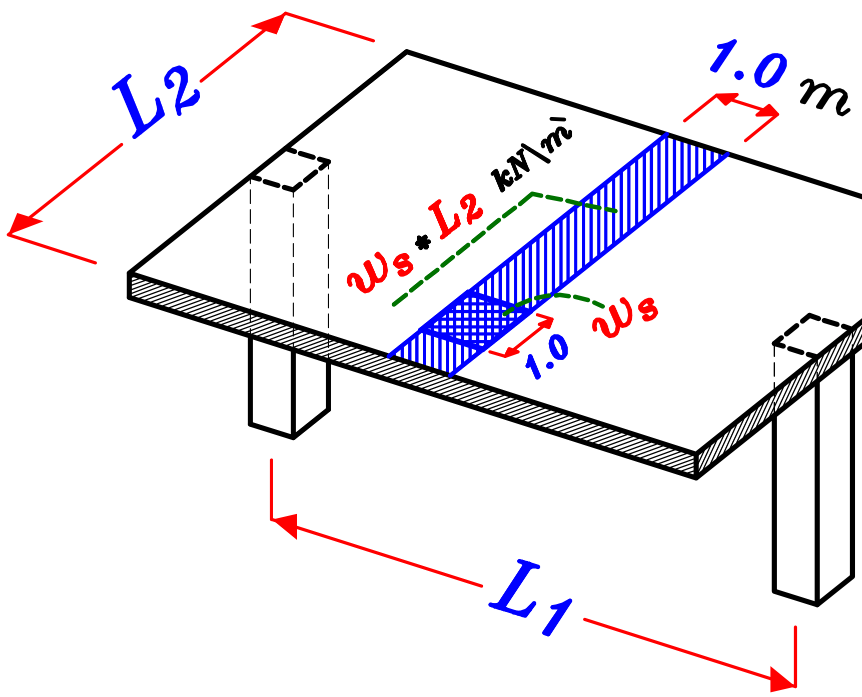
نختار محور للاعمده فى كلا من الاتجاهين (يفضل ان يكون المحور فى المنتصف)
و نأخذ شريحه فى اتجاهه يكون عرضها المسافه من C.L. البلاطه على
يمين المحور الى الـ C.L. البلاطه على يسار المحور .

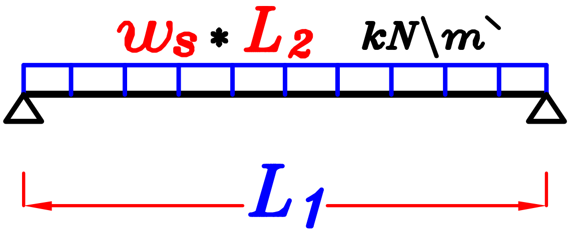
Strip at Long Direction.

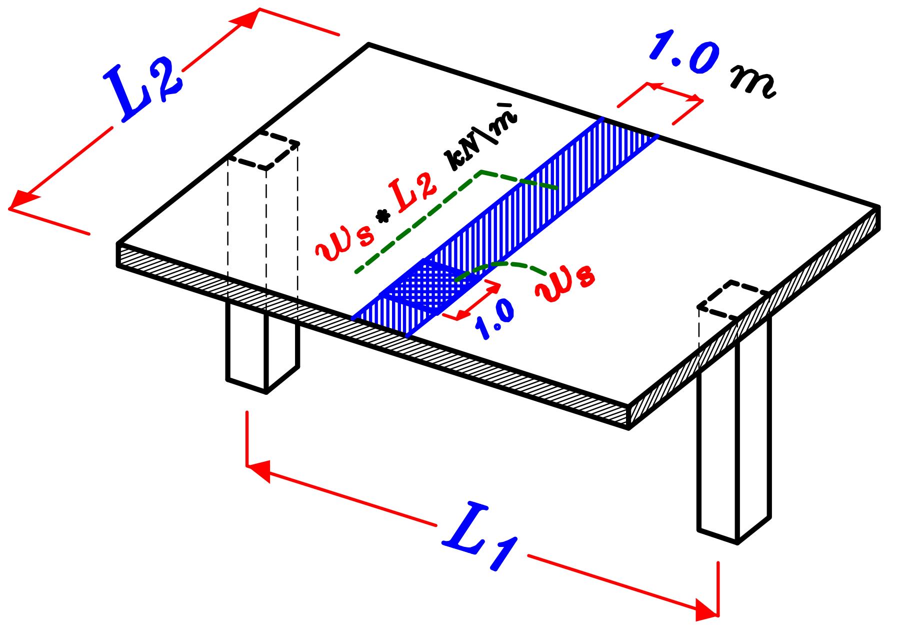


ثم نأخذ باكيه واحده فقط كأنها Simple Span

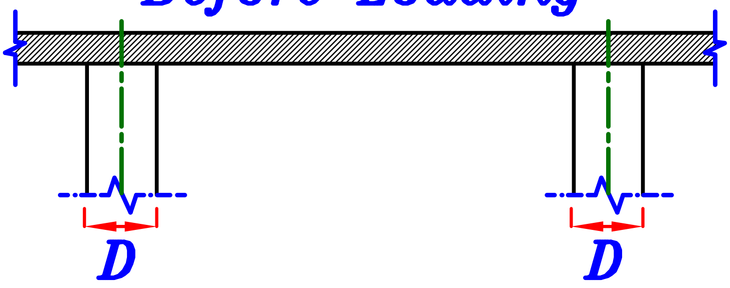
(اذا كانت الـ Spans اطوالها مختلفه نختار اكبر Span)



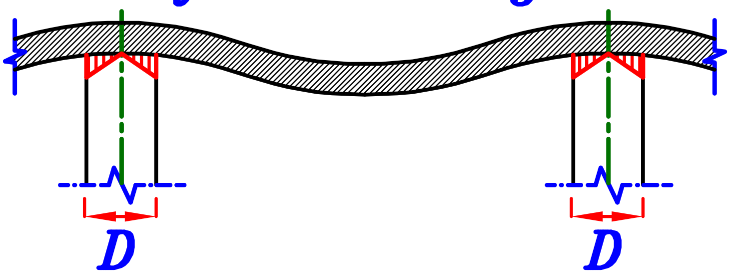


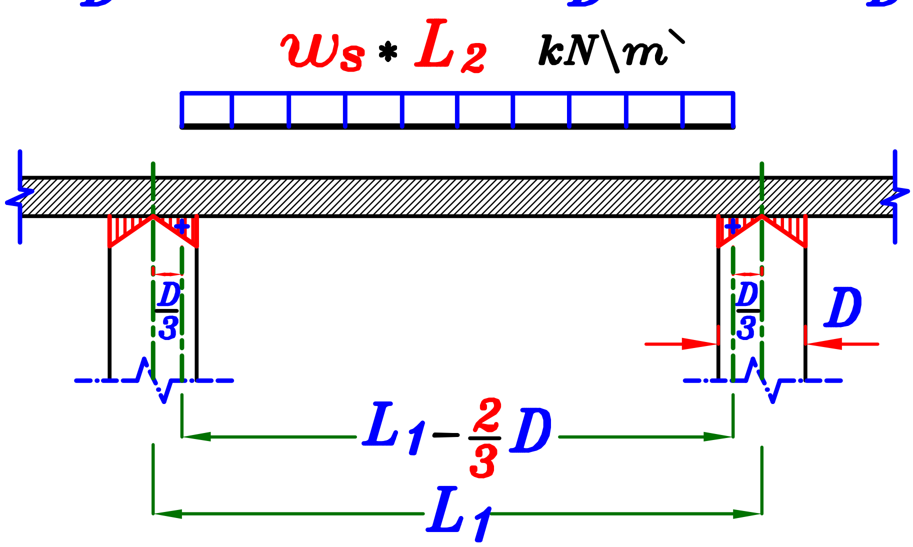


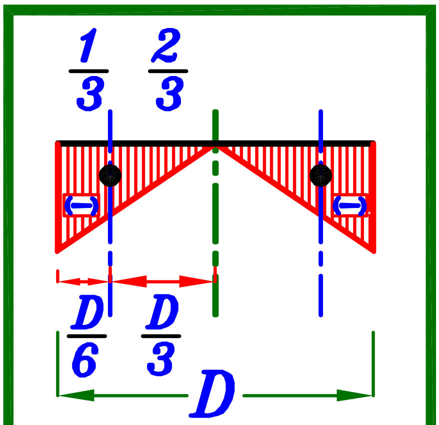
Before Loading



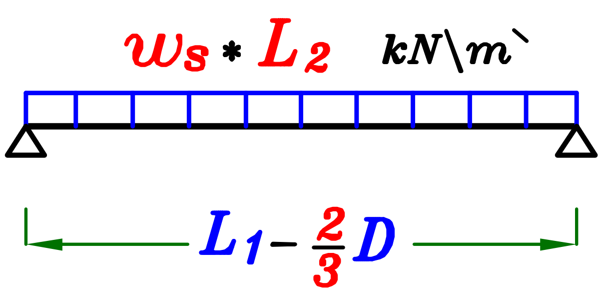
After Loading







Stress distribution on the section of the column

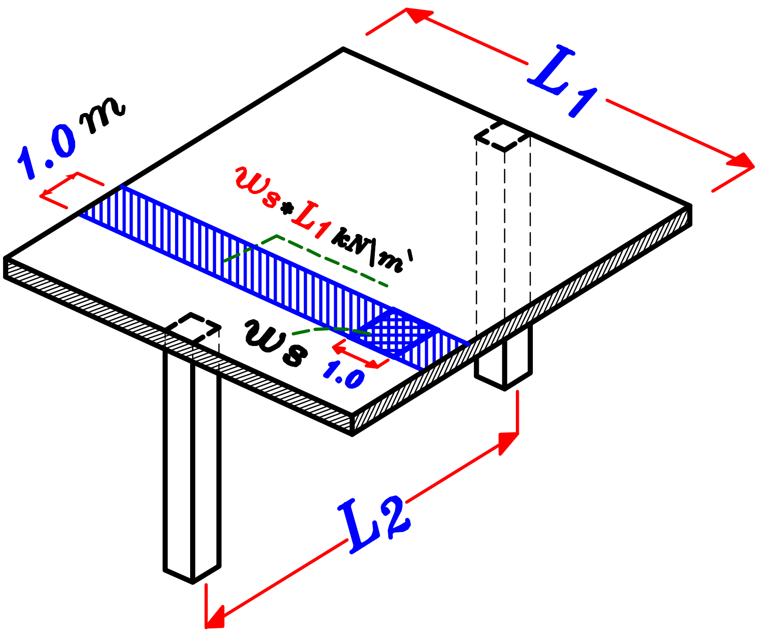


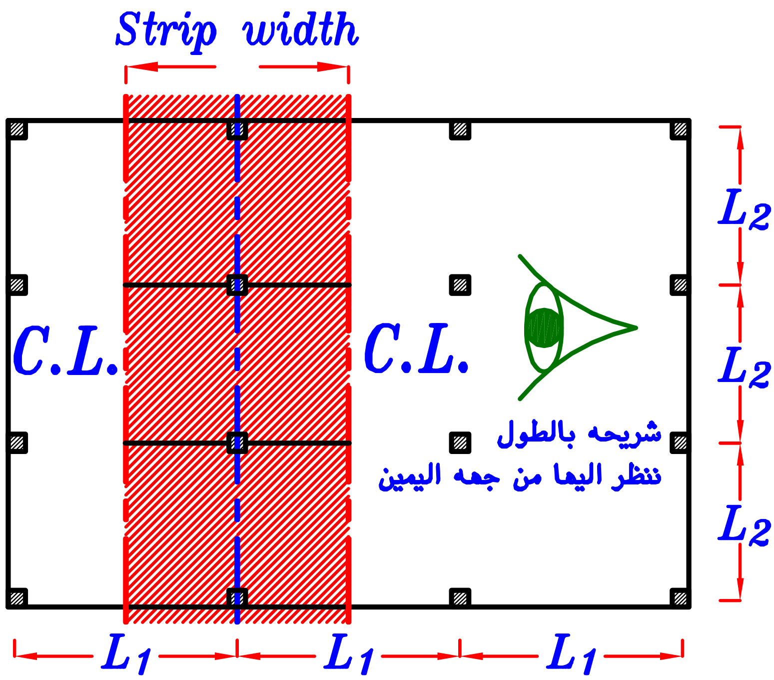
Moment at Long Direction.

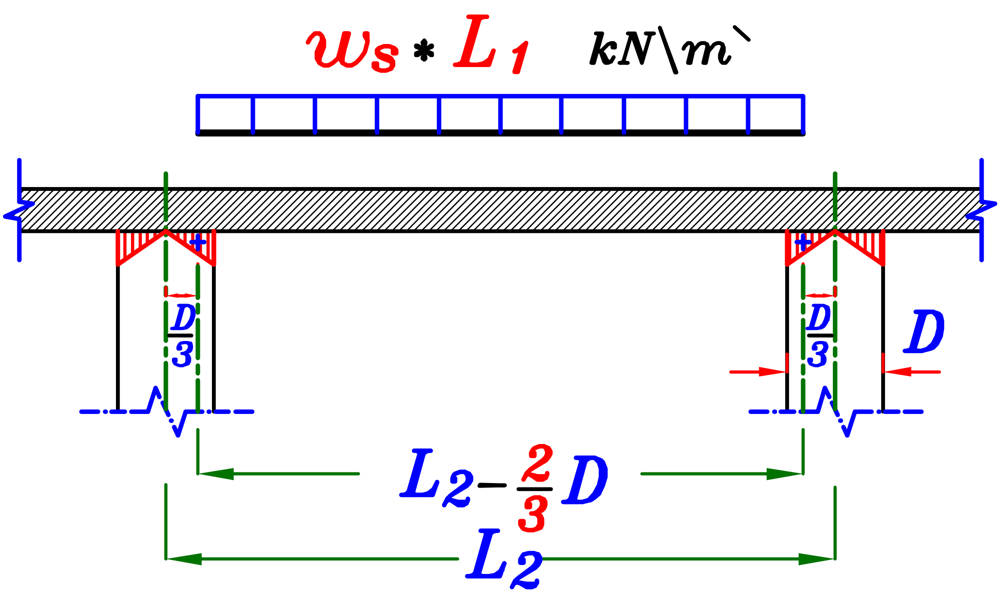
$$\frac{w L^2}{8}$$

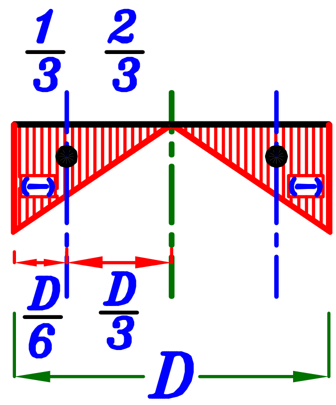
$$\boxed{M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3} D\right)^2}{8}}$$

Strip at Short Direction.

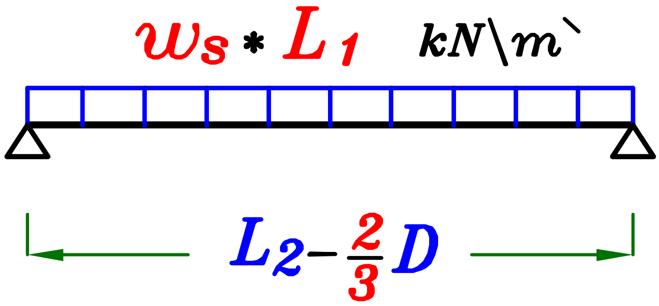








Stress distribution on the section of the column



Moment at Long Direction.

$$\frac{w\,L^2}{8}$$

$$\boxed{M_o = \frac{(w_s * L_1)\left(L_2 - \frac{2}{3}D\right)^2}{8}}$$

5- Distribute the B.M. (M_o) on C.S. & F.S.

يوزع الـ moment (M_o) على الشريحتين Column Strip & Field Strip

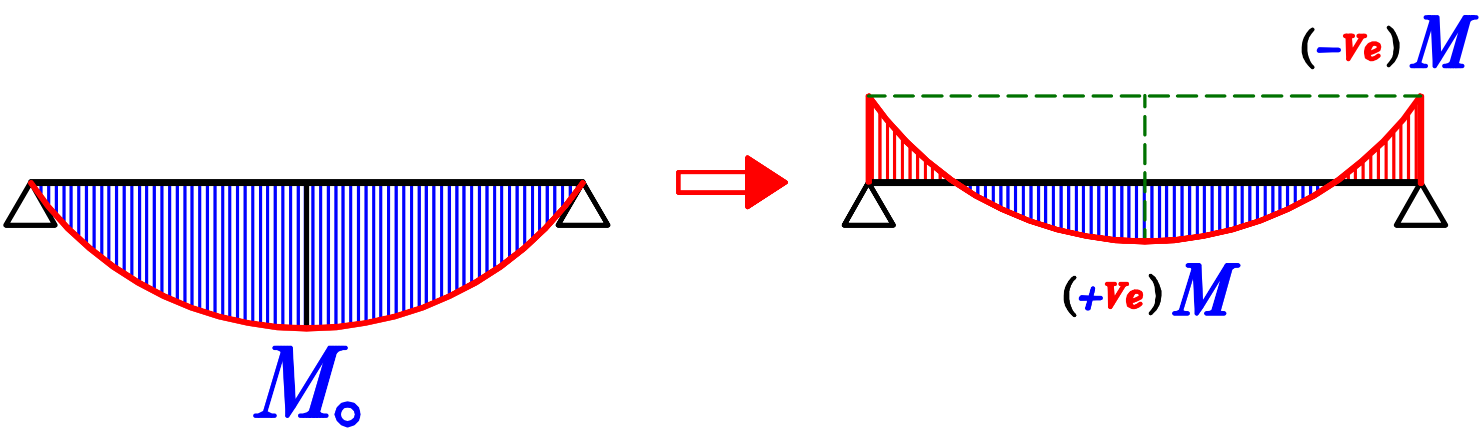
---

بما أن الـ moment (M_o) محسوب على اساس ان الشريحه Simple

و فى الحقيقه الشريحه Continuous اى يوجد بها moment (−Ve) و moment (+Ve)

اذاً يتم توزيع قيمه الـ (M_o) على كلا من الـ moment (−Ve) و moment (+Ve)

حوالى 60 % للـ moment (−Ve) و حوالى 40 % للـ moment (+Ve)

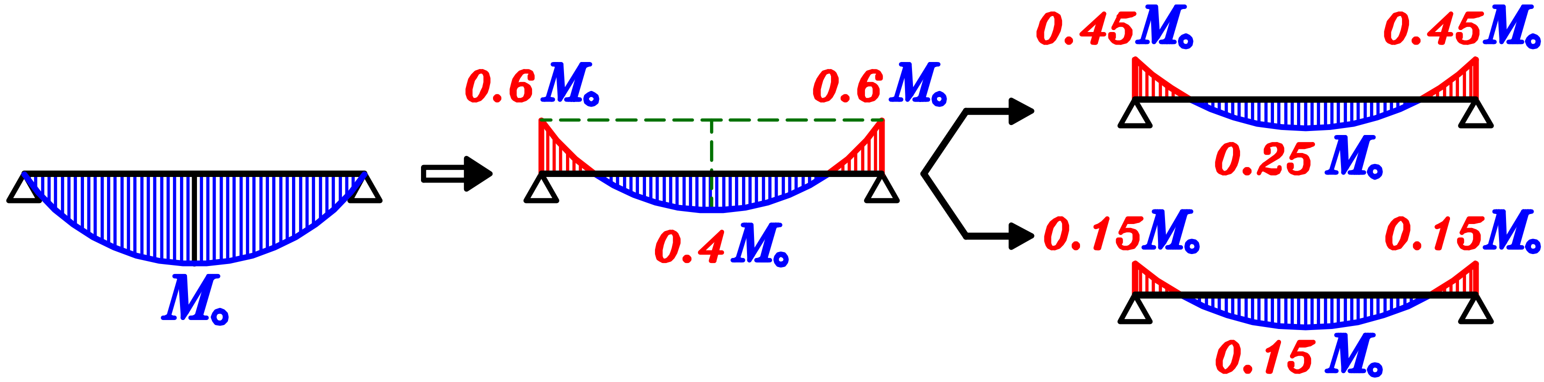


ثم يتم توزيع الـ moment (−Ve) على كلا من Column Strip & Field Strip

بنسب محفوظه 75 % للـ Column Strip و 25 % للـ Field Strip

ثم يتم توزيع الـ moment (+Ve) على كلا من Column Strip & Field Strip

بنسب محفوظه 62.5 % للـ Column Strip و 37.5% للـ Field Strip



يوزع ال moment (M_o) على الشريحتين Column Strip & Field Strip

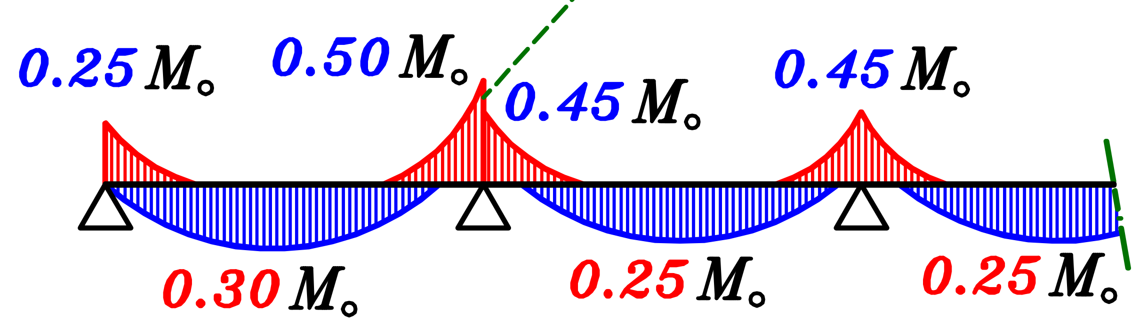
و يوجد عده حالات :

١ عرض ال F.S. يساوى عرض ال C.S. يساوى $\frac{L_2}{2}$

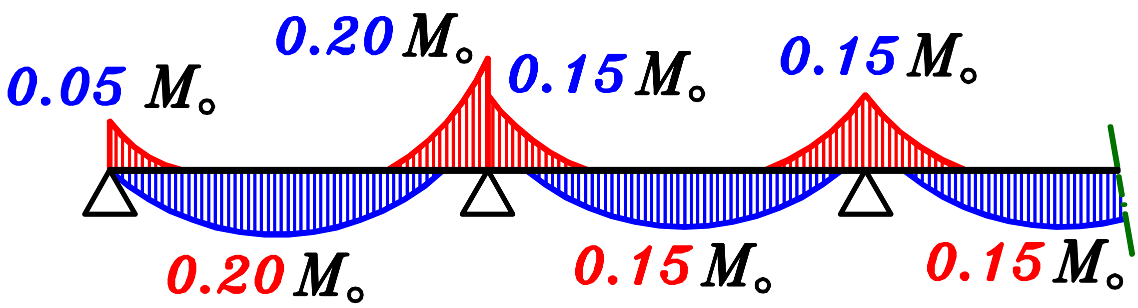
① Without Marginal Beam.

حفظ

يوجد فرق فى العزم عند أول باكيه فقط و الفرق يذهب الى العمود



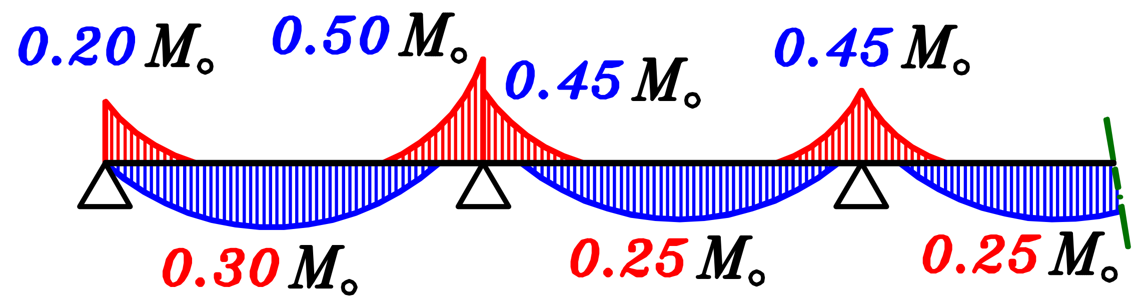
Column Strip ($M_{c.s.}$)



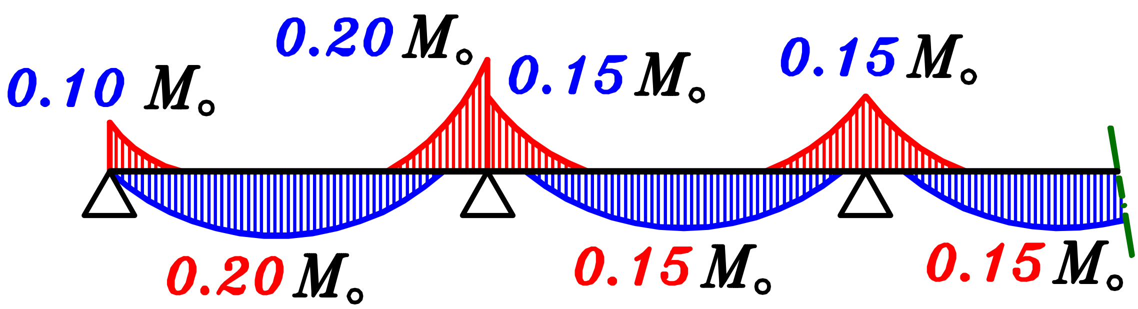
Field Strip ($M_{F.S.}$)

② With Marginal Beam.

حفظ

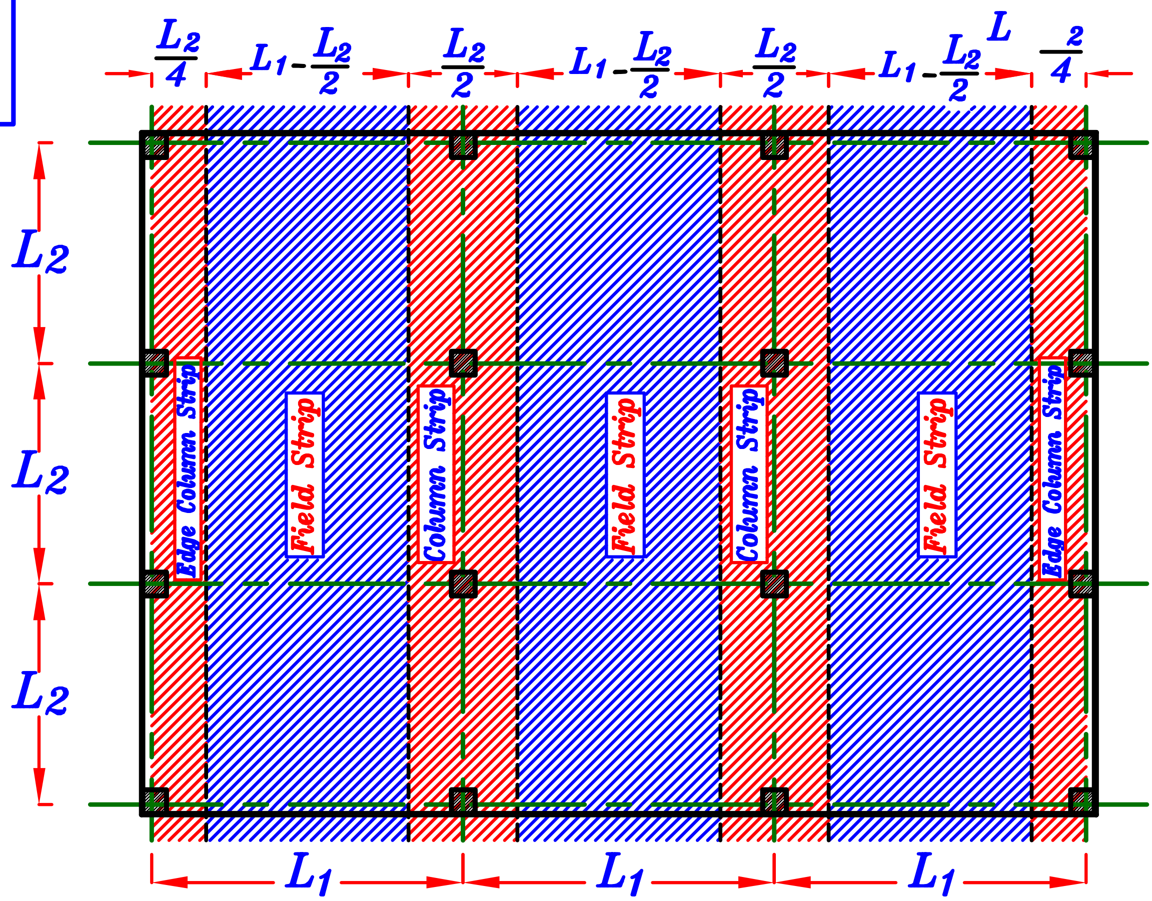


Column Strip ($M_{c.s.}$)



Field Strip ($M_{F.S.}$)

٢ عرض الـ F.S. مختلف عن عرض الـ C.S.



قيم الـ **Empirical moments** محسوبه على أساس أن عرض الـ **F.S.** يساوى عرض الـ **C.S.** يساوى نصف عرض الشريحه الكلى لذلك عند اختلاف عرض الـ **F.S.** عن عرض الـ **C.S.**

يتم ضرب عزم الـ **F.S.** فى **Modification Factor**

$$\boxed{Modification\ Factor = \frac{\text{العرض الحقيقى لـ Field Strip}}{\text{نصف العرض الكلى للشريحه من C.L. الى C.L.}}}$$

$$\boxed{Modification\ Factor = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2}}$$

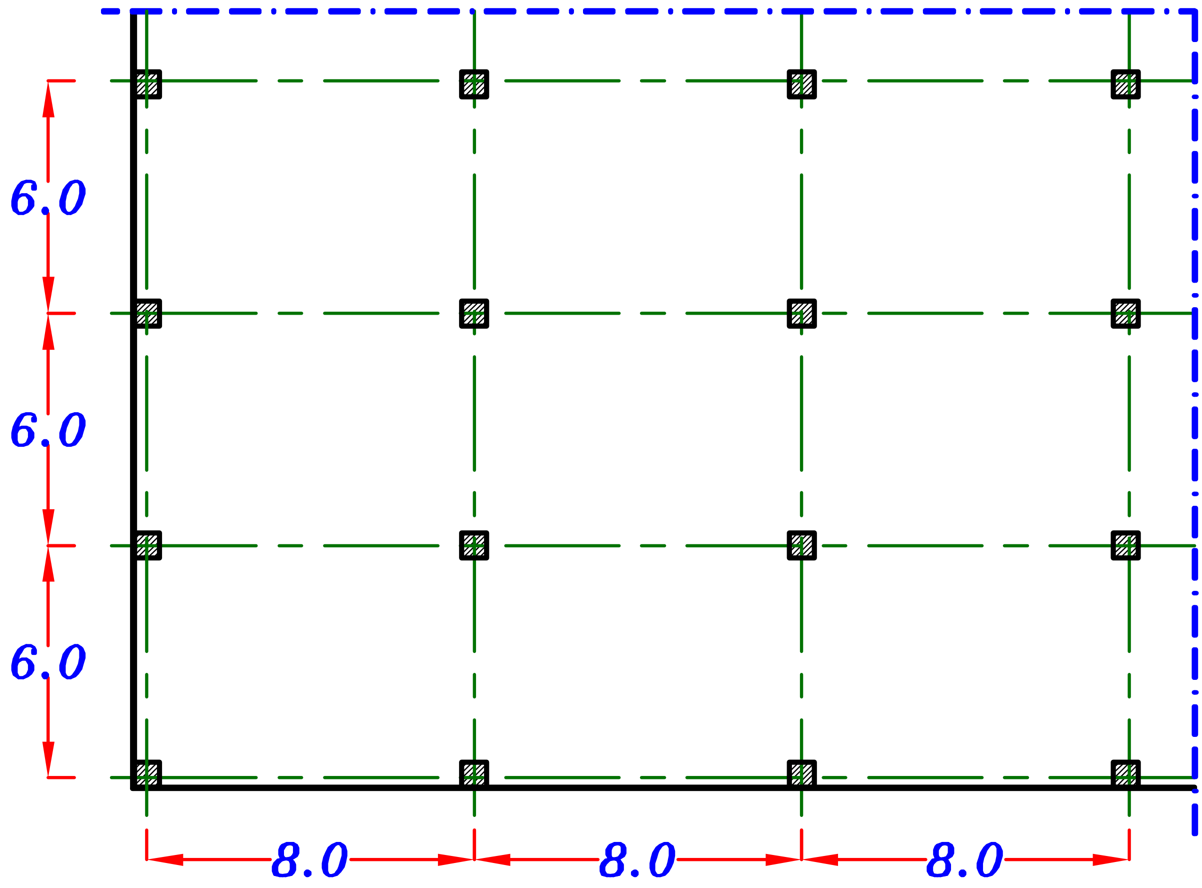
$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * Modification\ Factor}$$

ثم يتم اعاده حساب عزم الـ **C.S.** بحيث يظل العزم الكلى ثابت .

$$(M_{C.S.})_{mod.} + (M_{F.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.})$$

$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

Example.



IF $\boxed{M_o = 800 \text{ kN.m}}$ at Long Direction

IF $\boxed{M_o = 600 \text{ kN.m}}$ at Short Direction

Draw B.M.D. in both Column Strip & Field Strip at the two directions Using Empirical Values.

Solution.

$L_1 = 8.0\ m \qquad L_2 = 6.0\ m$

$\therefore$ Width of the Column Strip at the two directions $= \dfrac{L_2}{2} = \dfrac{6.0}{2} = 3.0\ m$

1 – For Long Direction.

$$\boxed{M_o = 800\ kN.m}$$

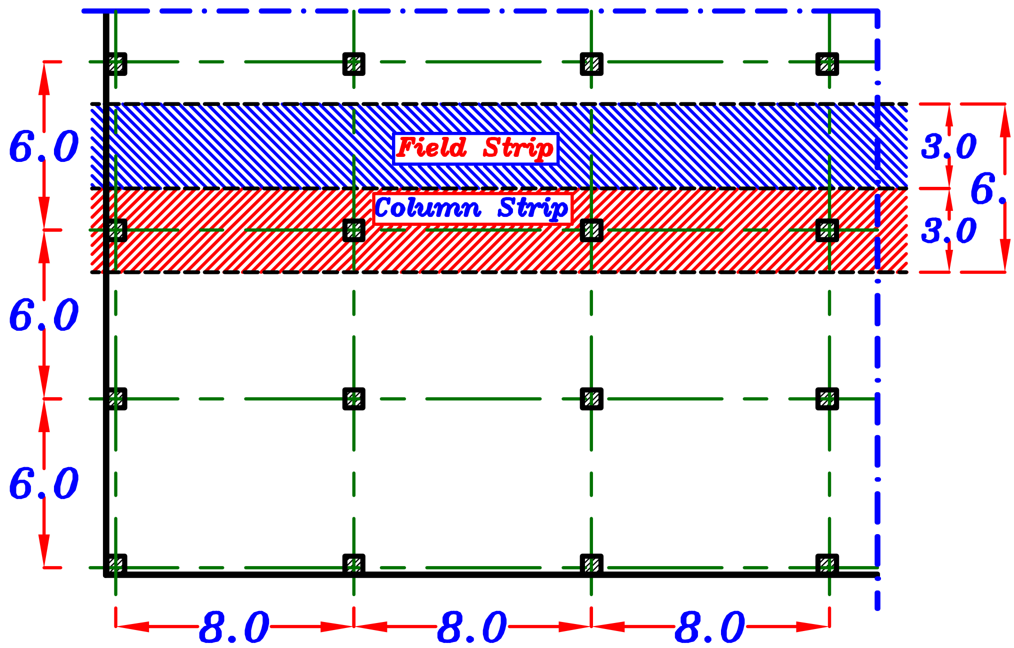
$\because$ Width of Column Strip $= 3.0\ m$

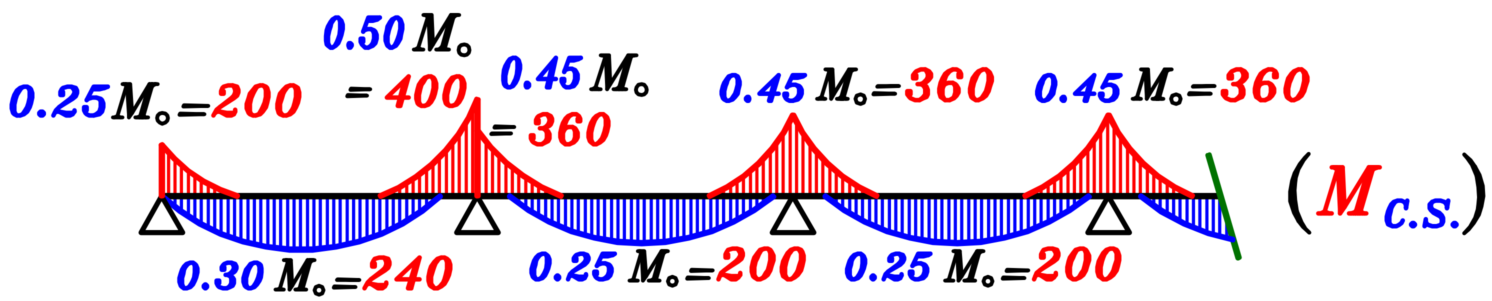
$\therefore$ Width of Field Strip = باقى عرض الشريحة كلها $= 6.0 - 3.0 = 3.0\ m$

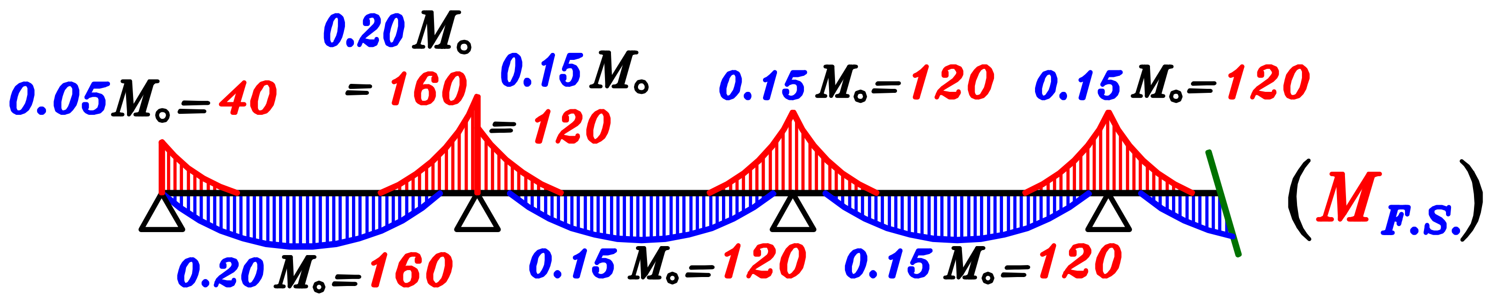
$\therefore$ Width of Column Strip = Width of Field Strip

$\therefore$ No Modification Factor.

$$\boxed{M_o = 800\ kN.m}$$







2- For Short Direction.

$M_o = 600 \ kN.m$

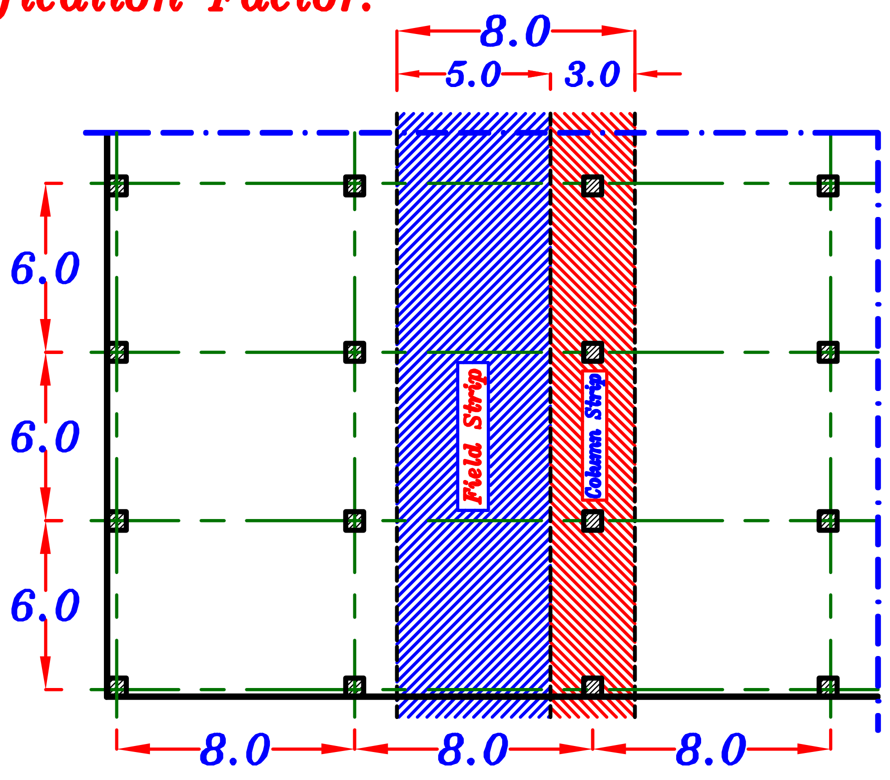
∵ Width of Column Strip = 3.0 m

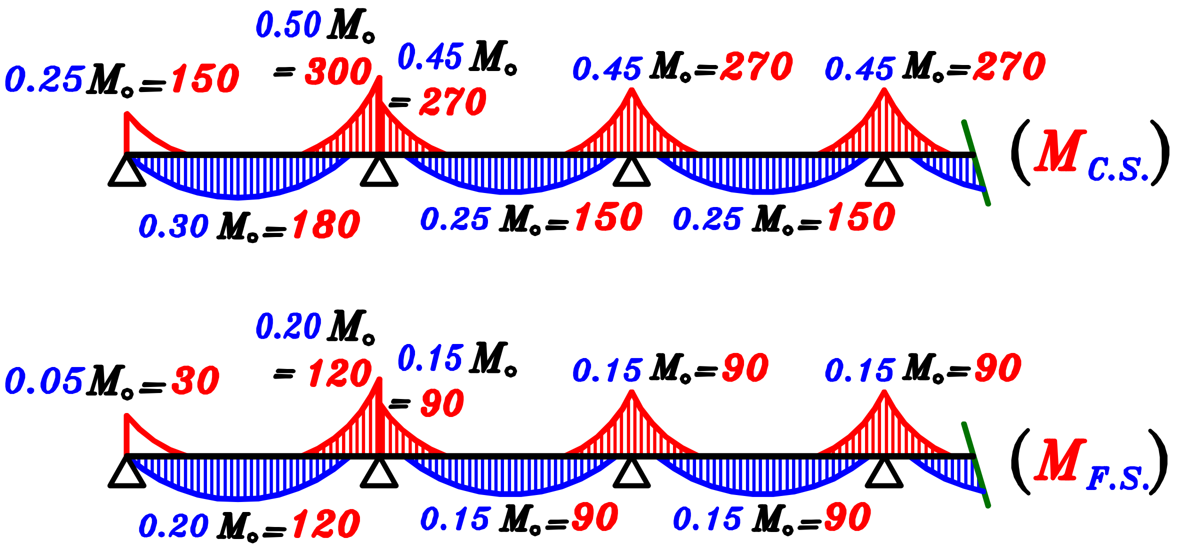
∴ Width of Field Strip = باقى عرض الشريحة كلها = 8.0 − 3.0 = 5.0 m

∴ Width of Column Strip ≠ Width of Field Strip

∴ There will be a Modification Factor.

$M_o = 600 \ kN.m$

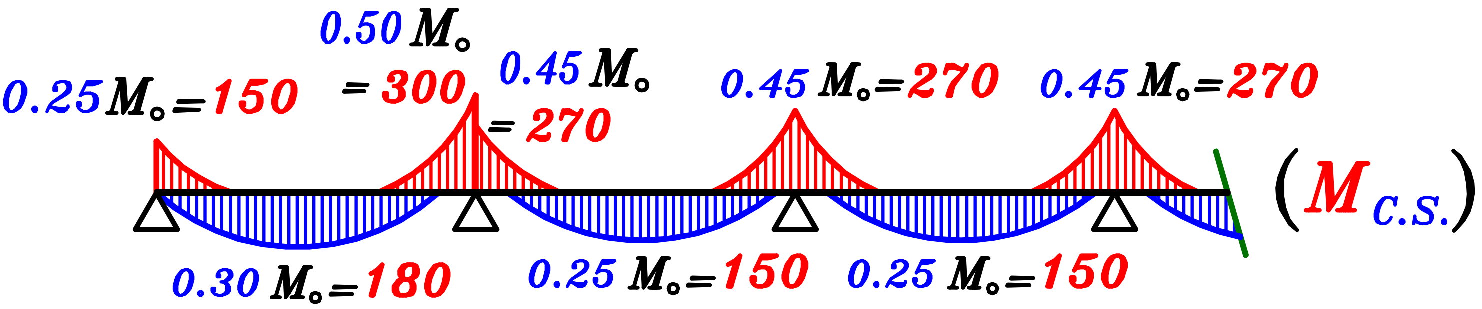


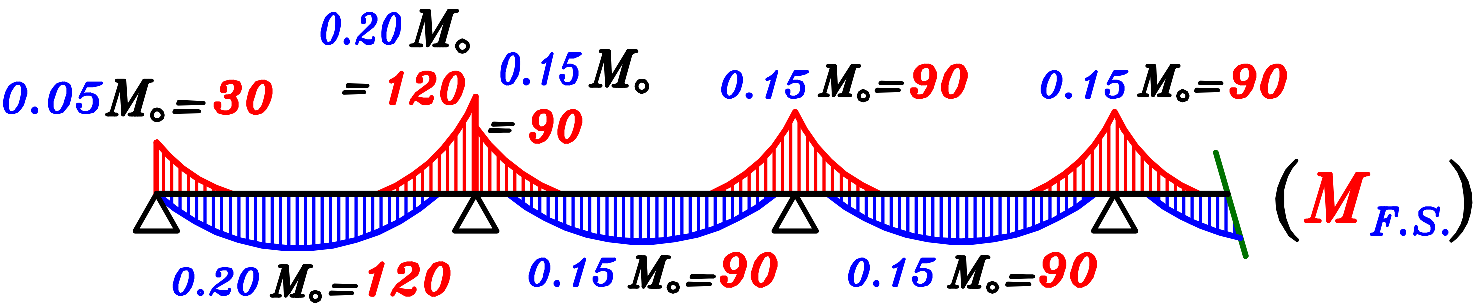


هذه العزوم محسوبه على أساس

$b_{C.S.} = b_{F.S.} = \frac{8.0}{2} = 4.0$

$$\text{Modification Factor} = \frac{\text{العرض الحقيقى لل Field Strip}}{\text{نصف العرض الكلى للشريحه من C.L. الى C.L.}} = \frac{5.0}{4.0}$$

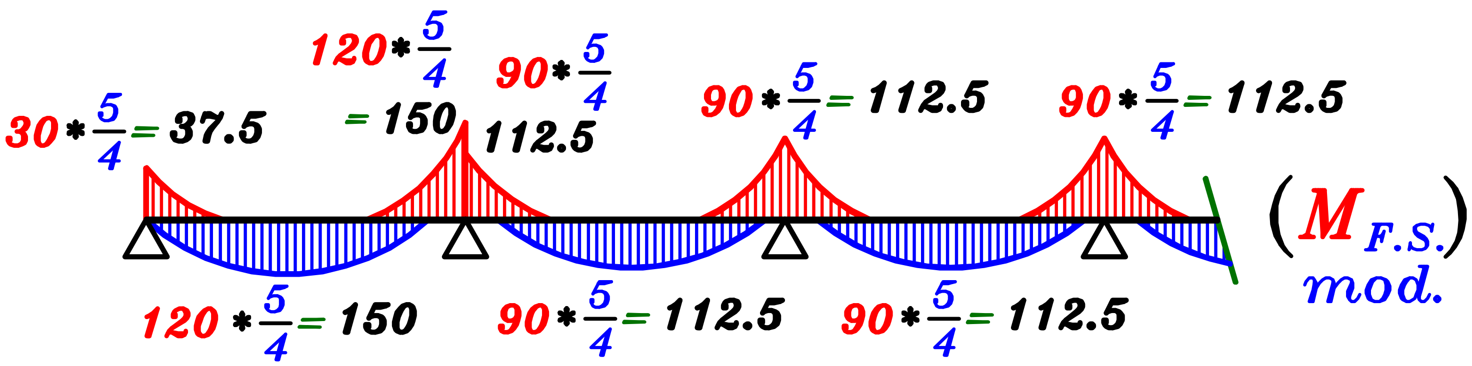




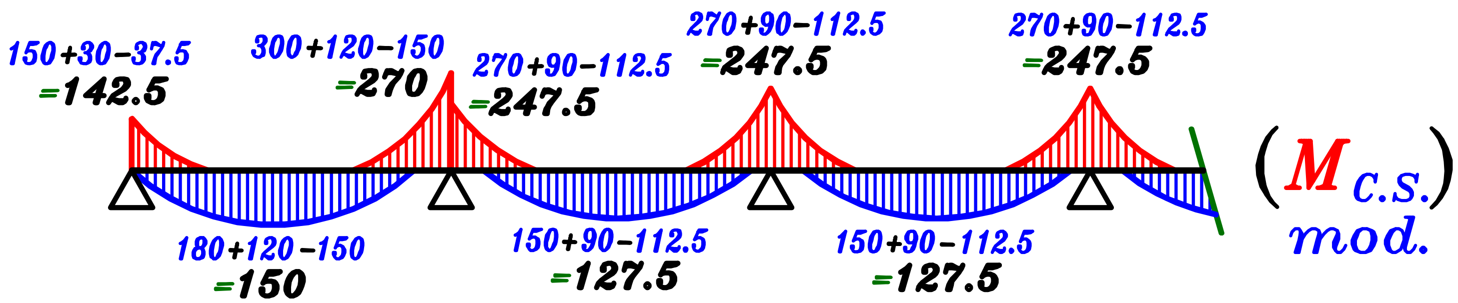
$$\text{Modification Factor} = \frac{\text{Field Strip العرض الحقيقى لل}}{\text{C.L. الى C.L. نصف العرض الكلى للشريحه من}} = \frac{5.0}{4.0}$$

Modified Moment.

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor}}$$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$



6- Design the strips by using charts (C_1, J)

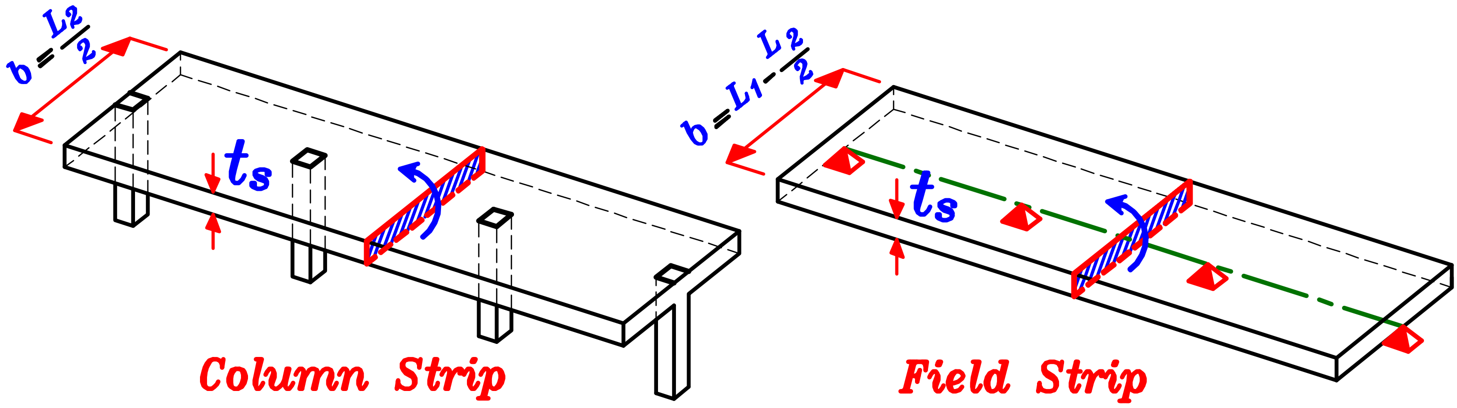
$$\boxed{d = C_1 \sqrt{\frac{M_{U.L.}}{F_{cu}\, b}}}$$

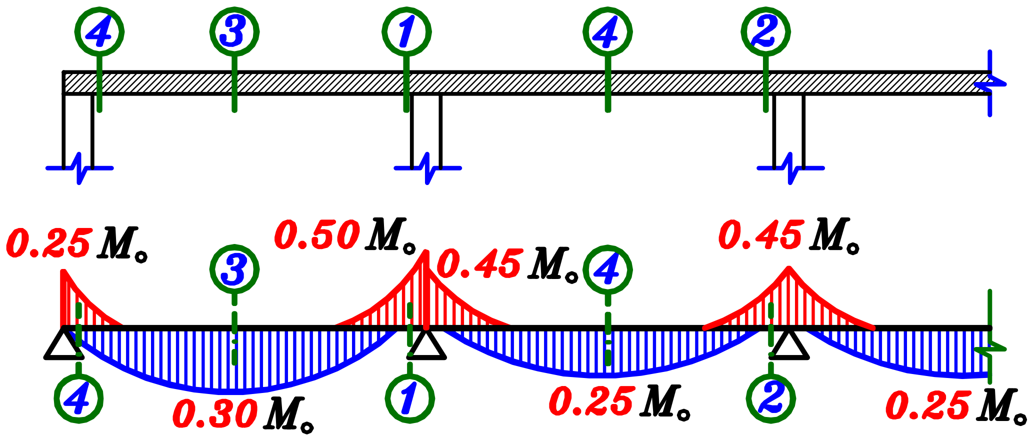
$M_{U.L.} = \checkmark$ kN.m\strip

b = Strip Width = عرض الشريحه

$d = t_s - cover$
- $t_s - 30$ mm فى الاتجاه الطويل (فرش)
- $t_s - 40$ mm فى الاتجاه القصير (غطاء)

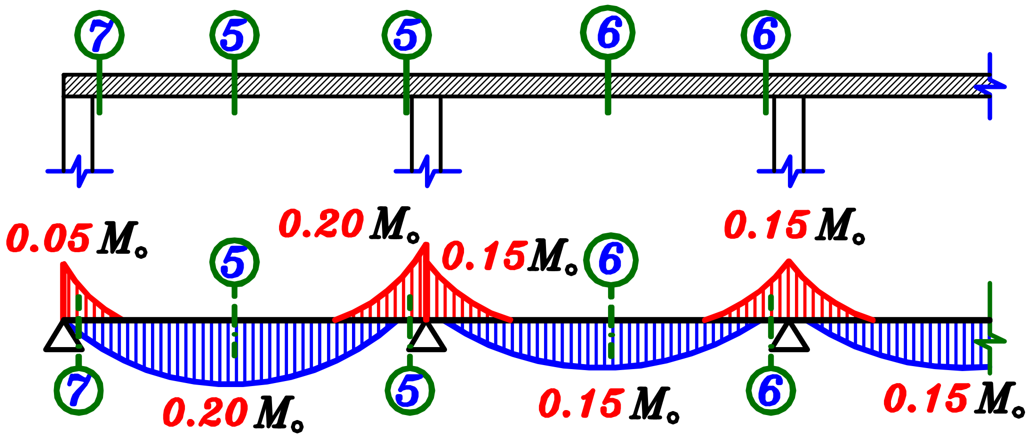
@ Without Drop Panel.





Column Strip

يفضل اختيار ال section أولا للعزم الاكبر ثم الذى يليه

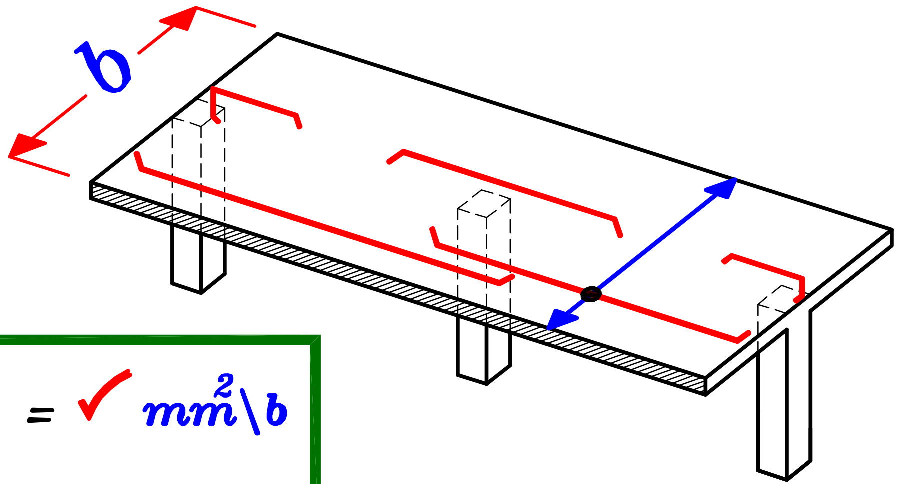


Field Strip

يفضل اختيار ال section أولا للعزم الاكبر ثم الذى يليه

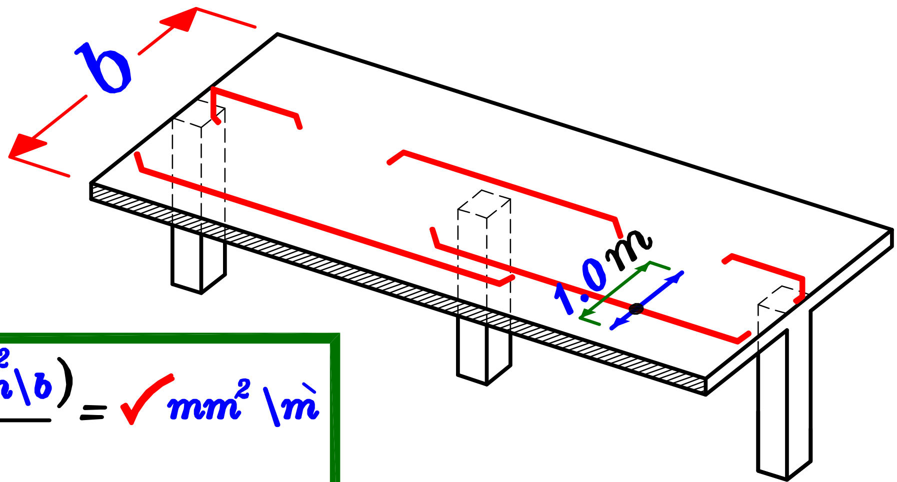
و يفضل عمل جدول لتصميم القطاعات كالاتى .

Strip	Sections	$M_{U.L.}$	B (mm)	d (mm)	C_1	J	A_s (total) (mm^2)	$A_s/m = \frac{A_s (total)}{b\ (m)} = (mm^2\backslash m)$
C.S.	1							
	2							
	3							
	4							
F.S.	5							
	6							
	7							



$$\therefore\quad A_s(mm^2\backslash b) = \frac{M_{U.L.}}{J\ F_y\ d} = \checkmark\ mm^2\backslash b$$

قيمه التسليح الموجود فى عرض الشريحه بالكامل.

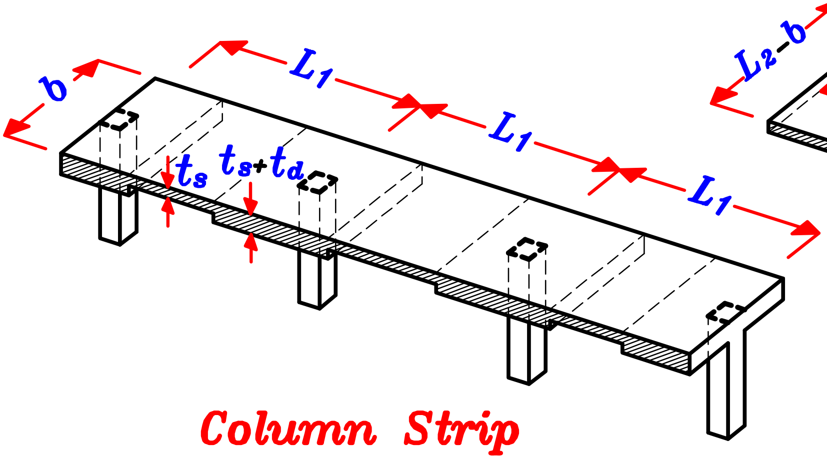


$$\therefore\quad A_s(mm^2\backslash m) = \frac{A_s(mm^2\backslash b)}{b} = \checkmark\ mm^2\backslash m$$

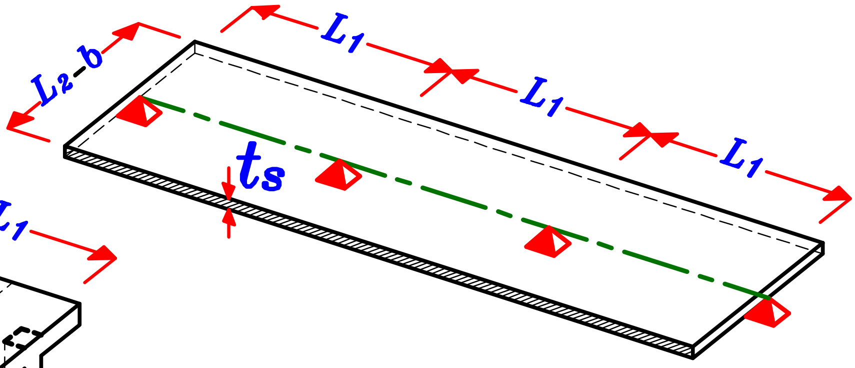
قيمه التسليح الموجود فى عرض ١,٠ م فقط.

$$A_{s_{min}} = 5\ \phi\ 12\backslash m$$

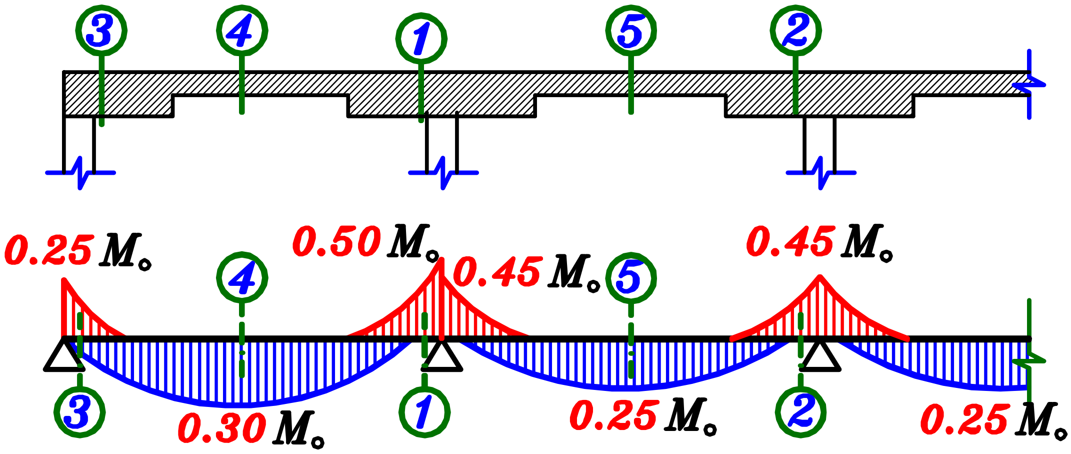
ⓑ With Drop Panel.



Column Strip

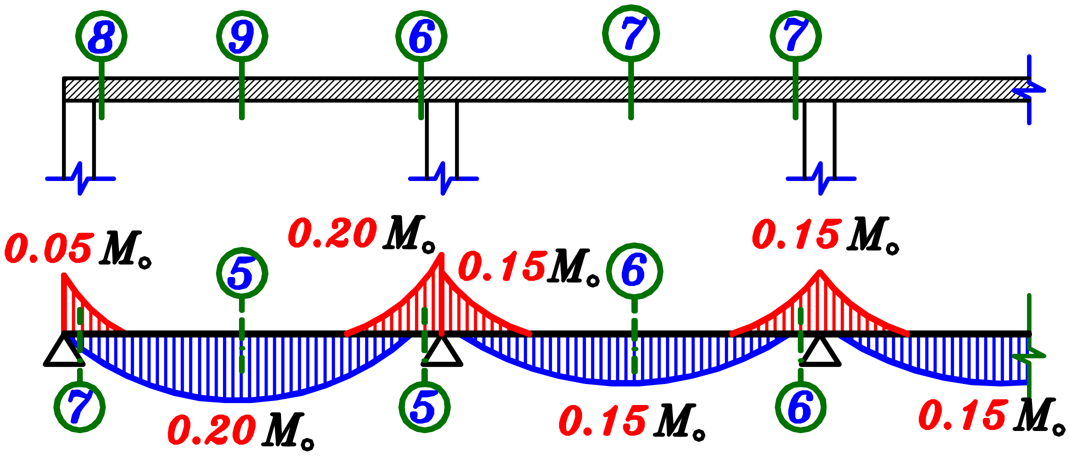


Field Strip



Column Strip

يفضل اختيار ال section
عند ال drop panel أولا



Field Strip

يفضل اختيار ال section
أولا للعزم الاكبر ثم الذى يليه

$$t = \begin{cases} t_s + t_d & \text{For sec ①, ② \& ③ in col strip.} \\ t_s & \text{For the all other sections.} \end{cases}$$

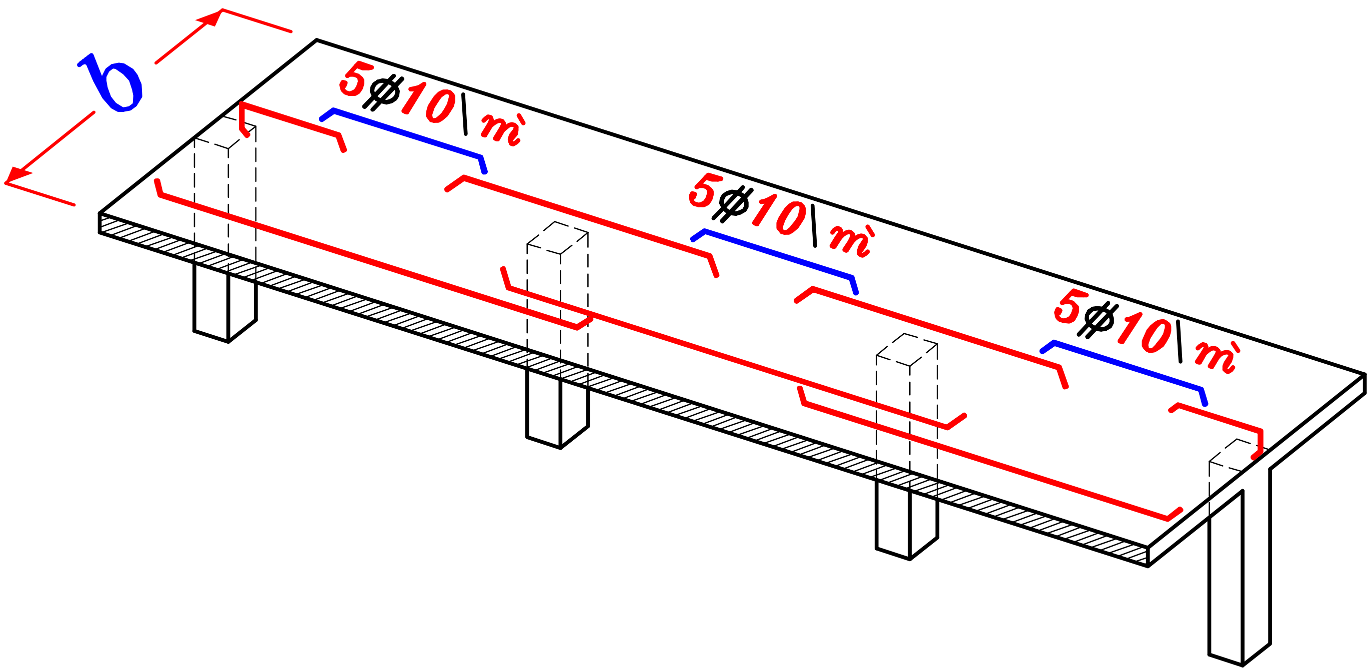
$$d = t - cover \begin{cases} t - 30 \text{ mm} & \text{فى الاتجاه الطويل (فرش)} \\ t - 40 \text{ mm} & \text{فى الاتجاه القصير (غطاء)} \end{cases}$$

و يفضل عمل جدول لتصميم القطاعات كالاتى .

Strip	Sections	$M_{U.L.}$	B (mm)	d (mm)	C_1	J	A_s (total) (mm^2)	$A_s/m = \frac{A_s\,(total)}{b\,(m)} = (mm^2\backslash m)$
C.S.	1							
	2							
	3							
	4							
	5							
F.S.	6							
	7							
	8							

ملحوظه

اذا كانت تخانه البلاطه أكبر من ١٦٠ مم نعمل شبكه علويه لمقاومه الانكماش وذلك بتكمله الحديد العلوى بحديد $5\varnothing10\backslash m`$

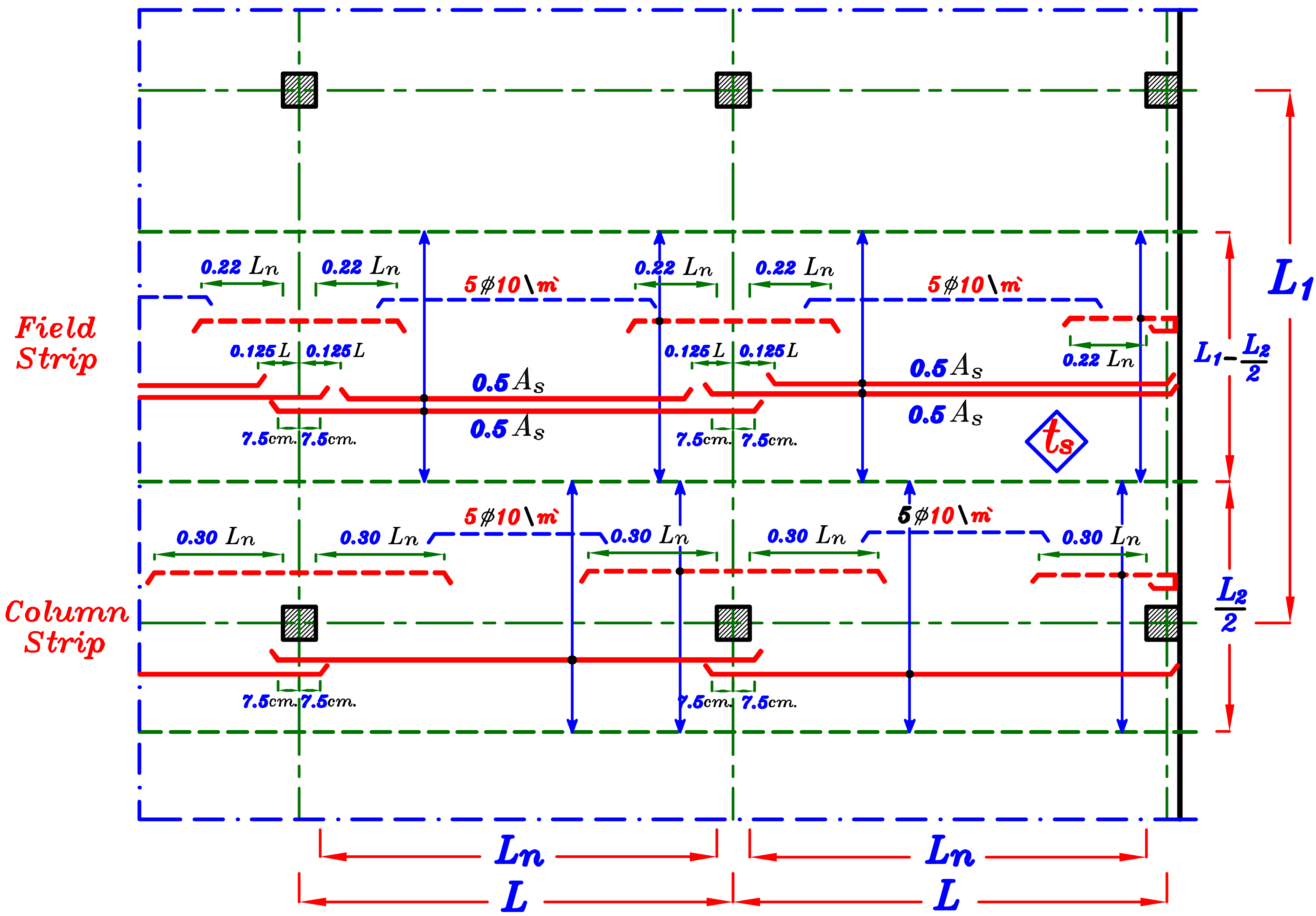


7- Draw details of RFT. in Plan.

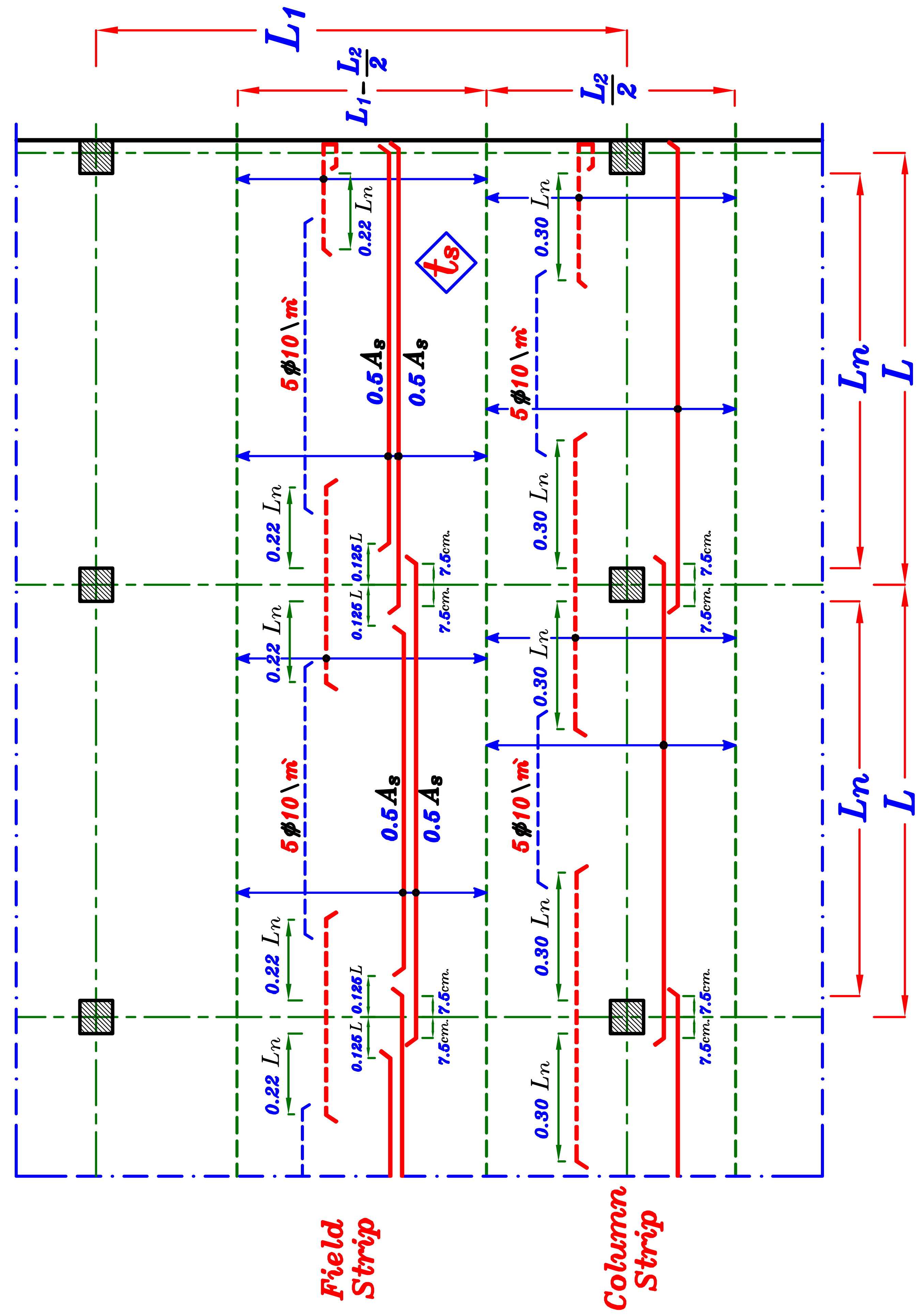
يفضل رسم الحديد العلوى بخط **dotted**

L_n هو الطول الخالص أى هو المسافه الداخليه من وجه العمود الى وجه العمود

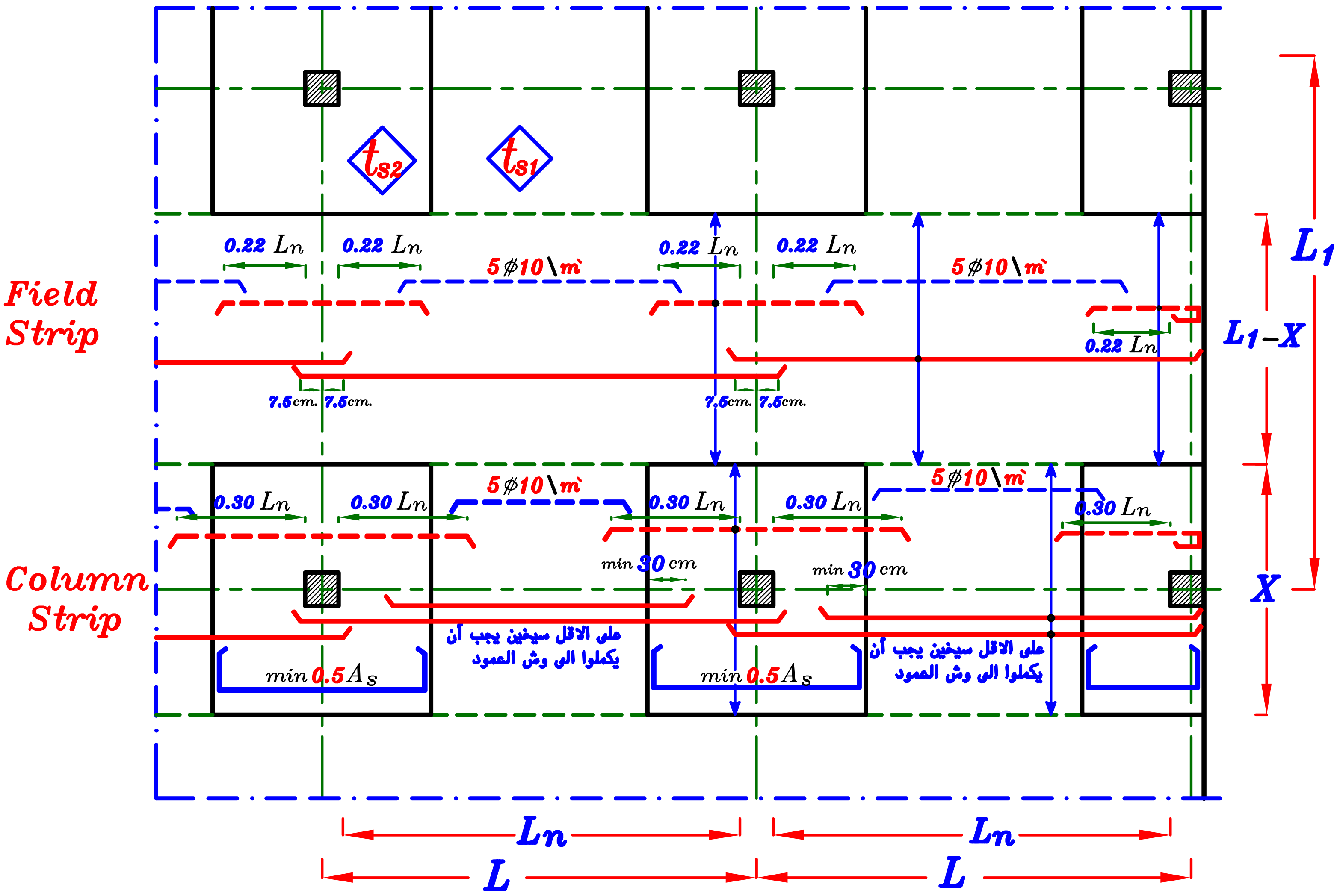
a - Without Drop Panel.



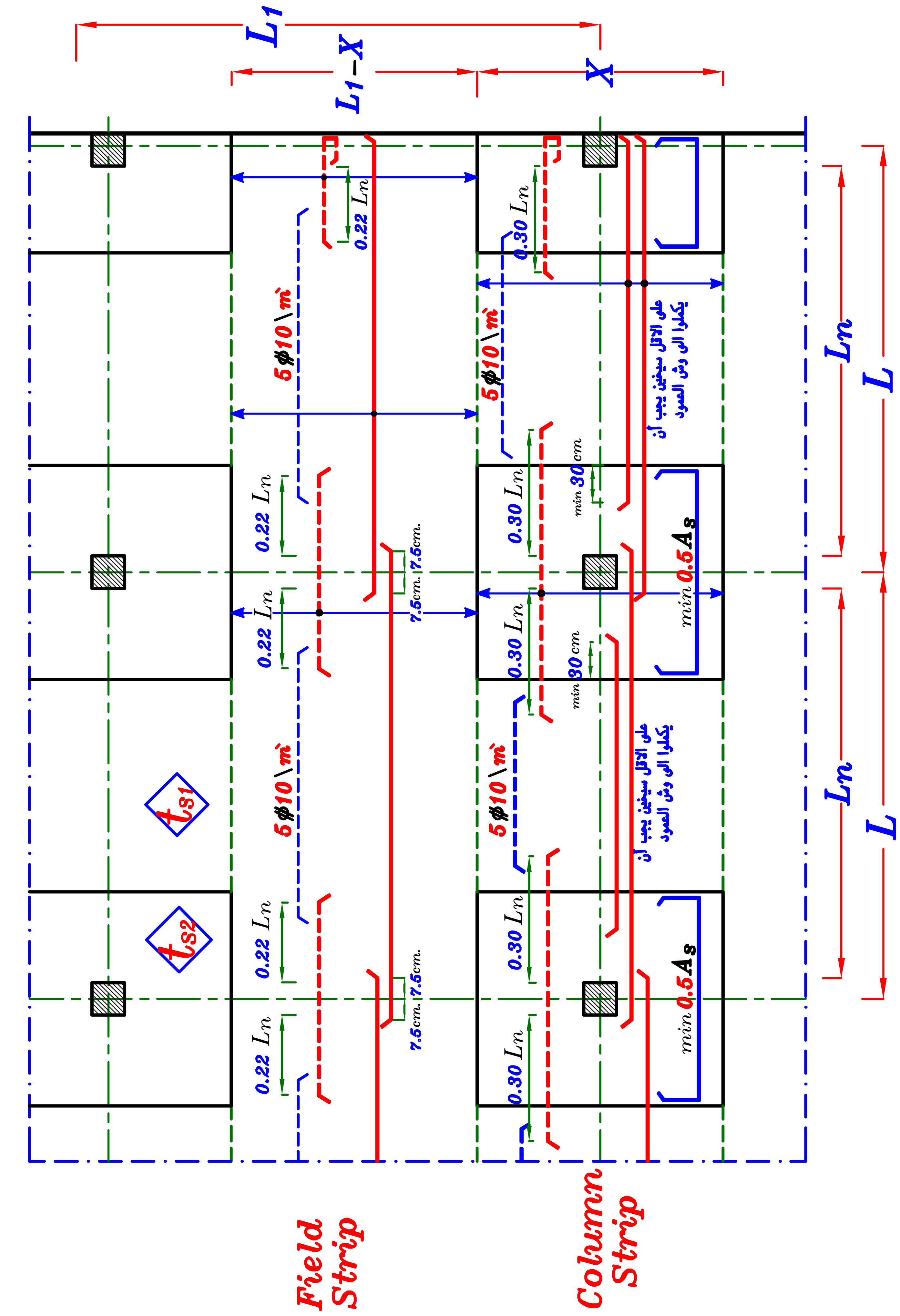
a - Without Drop Panel.



b – With Drop Panel.



b - With Drop Panel.

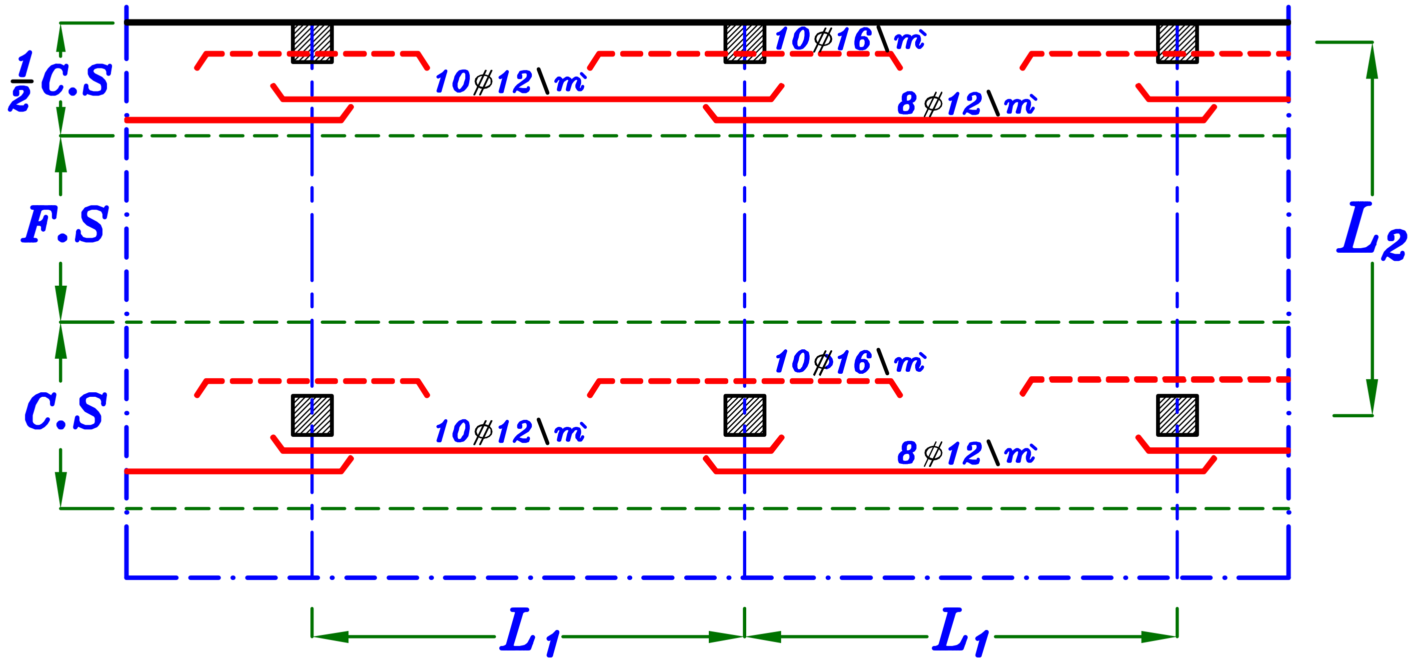


Reinforcement of Last Column Strip.

لتسليح الـ Column Strip الاخيره توجد حالتان :

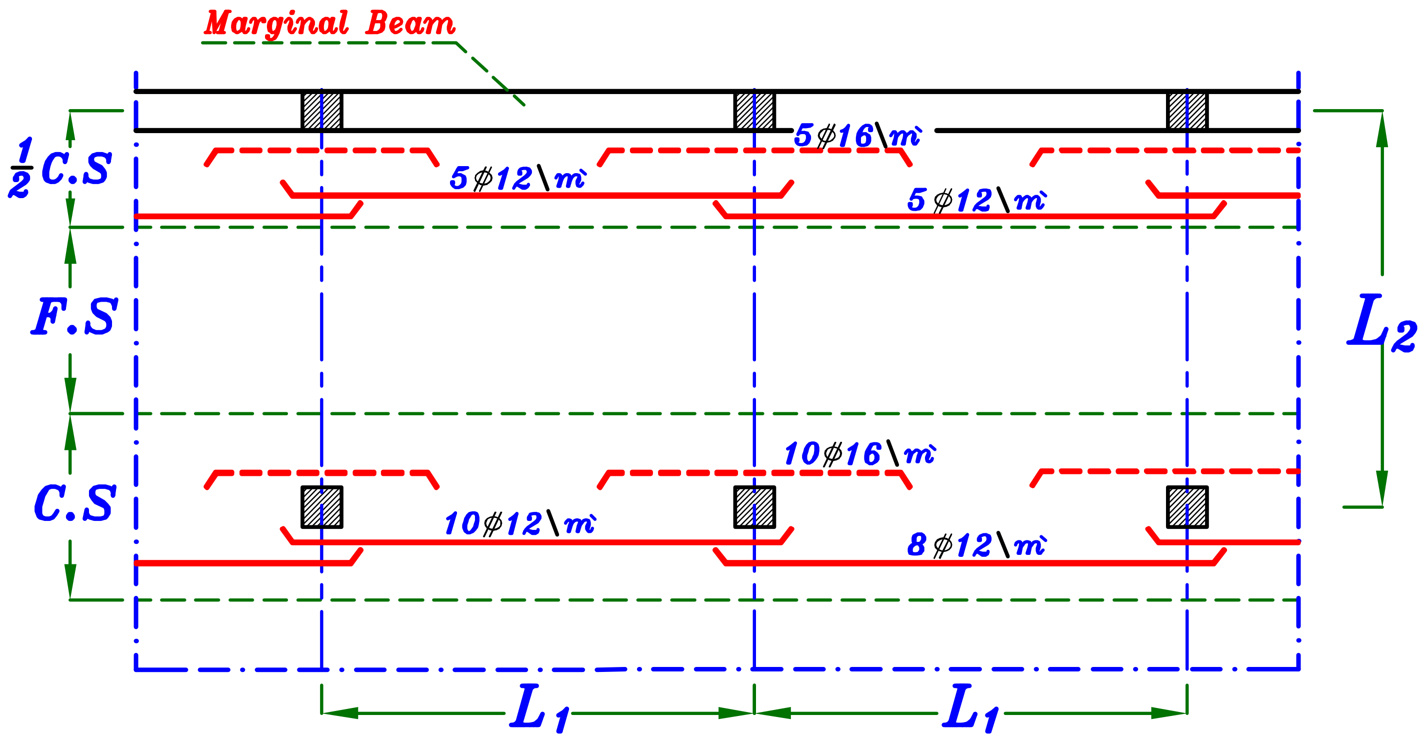
① Flat Slab without Marginal Beam.

الـ Column Strip التى فى الطرف يكون تسليحها فى المتر هو نفس تسليح الـ Column Strip الرئيسيه فى المتر .



② Flat Slab with Marginal Beam.

مساحه التسليح الموجود فى الـ Column Strip المجاوره لل Marginal Beam فى المتر الواحد تساوى نصف مساحه الحديد الموجود فى المتر لل Column Strip الرئيسيه



Example.

The given plan shows general layout of a Flat slab Floor

The column height 5.50 m

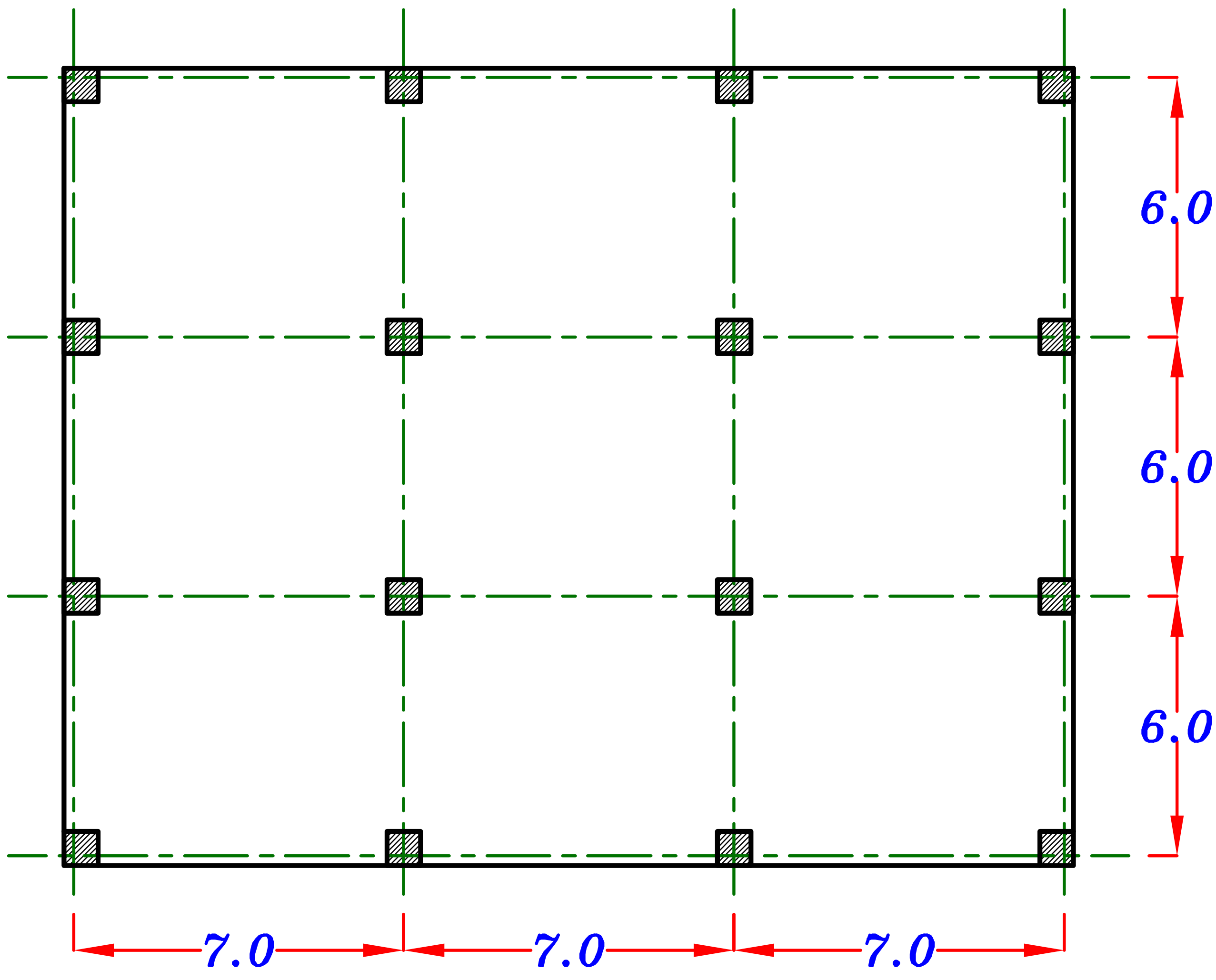
Data.

$F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

$F.C. = 1.5\ kN\backslash m^2$, $L.L. = 4.0\ kN\backslash m^2$, $Walls = 1.5\ kN\backslash m^2$

Req.

1. Using empirical method calculate the moments For both the Field strip and the column strip in both directions.
2. design the sections of the slab. and draw details of reinforcement in plan.



Solution.

1- Concrete Dimensions.

Column dimensions.

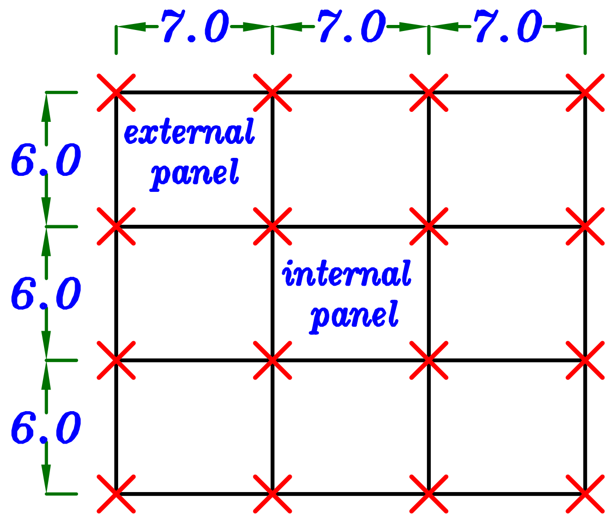
$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{5500}{15} = 366.7\ mm \\ \dfrac{L_1}{20} = \dfrac{7000}{20} = 350\ mm \end{cases}$$

$$\boxed{b_{Col.} = 400\ mm}$$

$(400 * 400)$

Slab Thikness.

$L_1 = 7.0\ m$



External panel $t_s = \dfrac{L_1}{32} = \dfrac{7000}{32} = 218.7\ mm$

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7000}{36} = 194.4\ mm$

$$\boxed{t_s = 220\ mm}$$

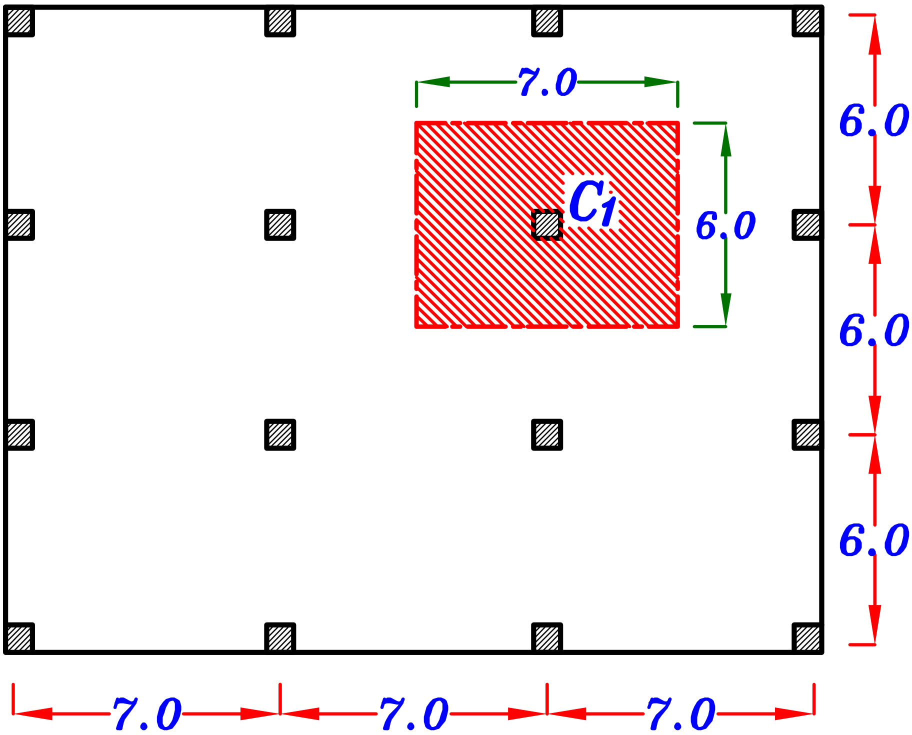
2- Loads on the Slab.

$$w_{s\,U.L.} = 1.4\,(t_s\,\gamma_c + F.C. + Walls) + 1.6\,(L.L.)$$

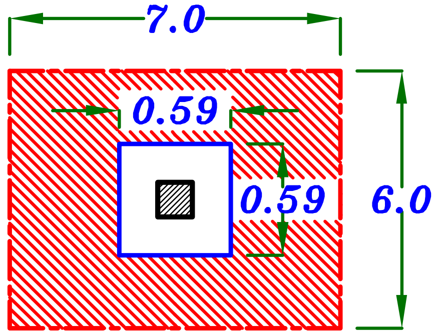
$$w_{s\,U.L.} = 1.4\,(0.22 * 25 + 1.50 + 1.50) + 1.6\,(4.0) = 18.30\ kN\backslash m^2$$

3 – Check Punching on interior column.

كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى



$\underline{\underline{C_1}}$ Interior Column.



$$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$$

$$C + d = 0.40 + 0.19 = 0.59\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 18.30\left[7.0 * 6.0 - 0.59 * 0.59\right] = 762.2\ kN$$

$$A_p = (b_o * d) = (4 * 590) * 190 = 448400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{762.2 * 10^3}{448400} * 1.15 = 1.95\ N/mm^2$$

$$q_{pcu} = 0.8\left(\frac{\alpha\, d}{b_o} + 0.2\right)\sqrt{\frac{F_{cu}}{\gamma_c}}$$

$$q_{pcu} = 0.8\left(\frac{4.0 * 190}{4 * 590} + 0.2\right)\sqrt{\frac{25}{1.5}} = 1.704\ N/mm^2$$

$$OR \quad q_{p_{cu}} = 0.316\,(0.5 + \frac{a}{b})\sqrt{\frac{F_{cu}}{\gamma_c}}$$

$$= 0.316\,(0.5 + \frac{0.4}{0.4})\sqrt{\frac{25}{1.5}} = 1.935 \ N/mm^2$$

$$OR \quad q_{p_{cu}} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29 \ N/mm^2$$

$$OR \quad q_{p_{cu}} = 1.60 \ N/mm^2$$

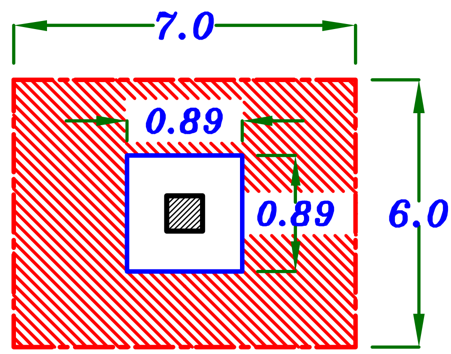
By taking the smaller value

$$\therefore \quad q_{p_{cu}} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29 \ N/mm^2$$

$$\boxed{q_{p_{cu}} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}}}$$ فى الامثله القادمه سنعتبر دائما ان

$$\therefore \quad \boxed{q_{pu} > q_{p_{cu}}} \longrightarrow$$ Unsafe punching
Increase dimensions of the column

C_1 Interior Column.



Take the Column (700 * 700)

$$d = t_s - 30\,mm = 220 - 30 = 190 \ mm = 0.19 \ m$$

$$C + d = 0.70 + 0.19 = 0.79 \ m$$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 18.30\left[7.0 * 6.0 - 0.89 * 0.89\right] = 754.1 \ kN$$

$$A_p = (b_o * d) = (4 * 890) * 190 = 676400 \ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{754.1 * 10^3}{676400} * 1.15 = 1.28 \ N/mm^2$$

$$q_{p_{cu}} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29 \ N/mm^2$$

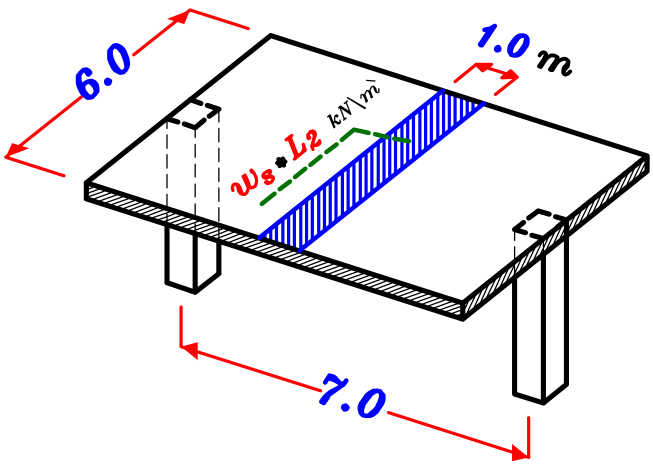
$$\boxed{q_{pu} < q_{p_{cu}}} \longrightarrow$$ Safe Punching.

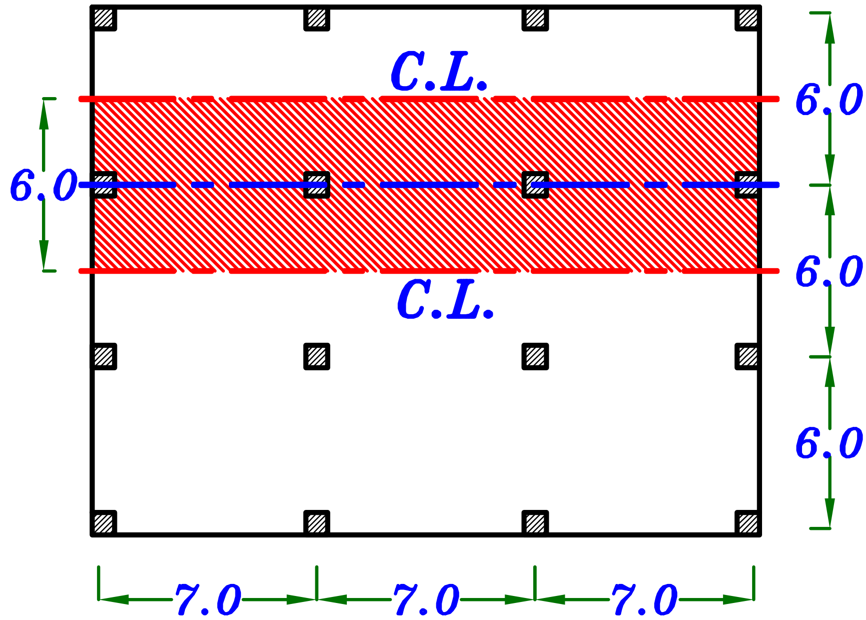
4- Take a Strips in the slabs at the long and short directions.
The strip width From C.L. the slab to C.L. the slab.
and Calculate the moment on the panel.

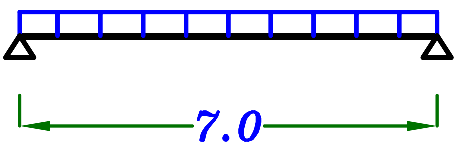
Strip at Long Direction.

Span $= 7.0\ m$

Width $= 6.0\ m$







Moment in Long Direction.

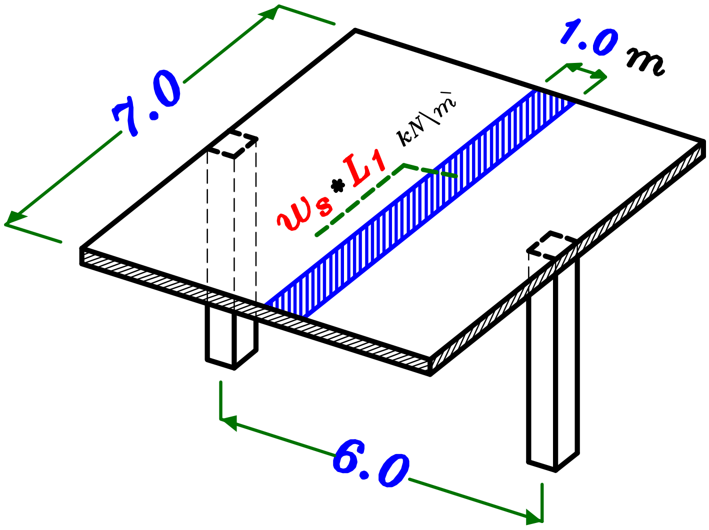
$$M_\circ = \frac{(w_s * L_2)\,(L_1 - \frac{2}{3} D)^2}{8} = \frac{(18.30 * 6.0)\,(7.0 - \frac{2}{3} * 0.7)^2}{8}$$

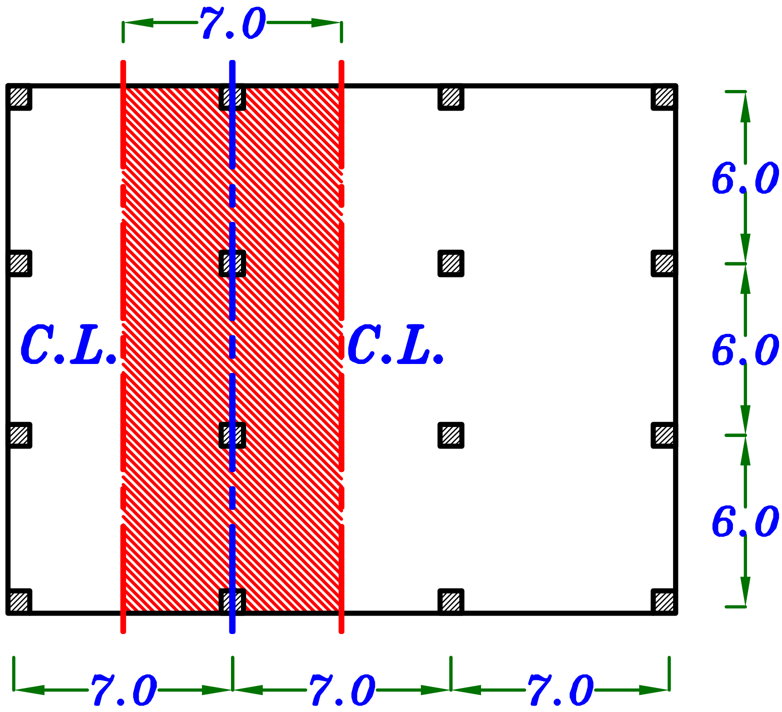
$$\boxed{M_\circ = 585.84\ kN.m}$$ Long Direction

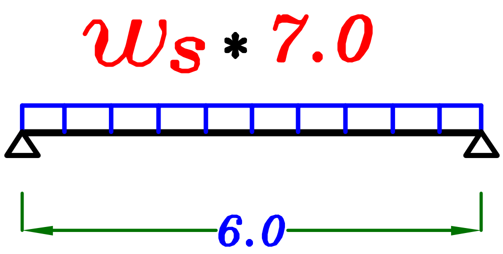
Strip at Short Direction.

Span $= 6.0\ m$

Width $= 7.0\ m$







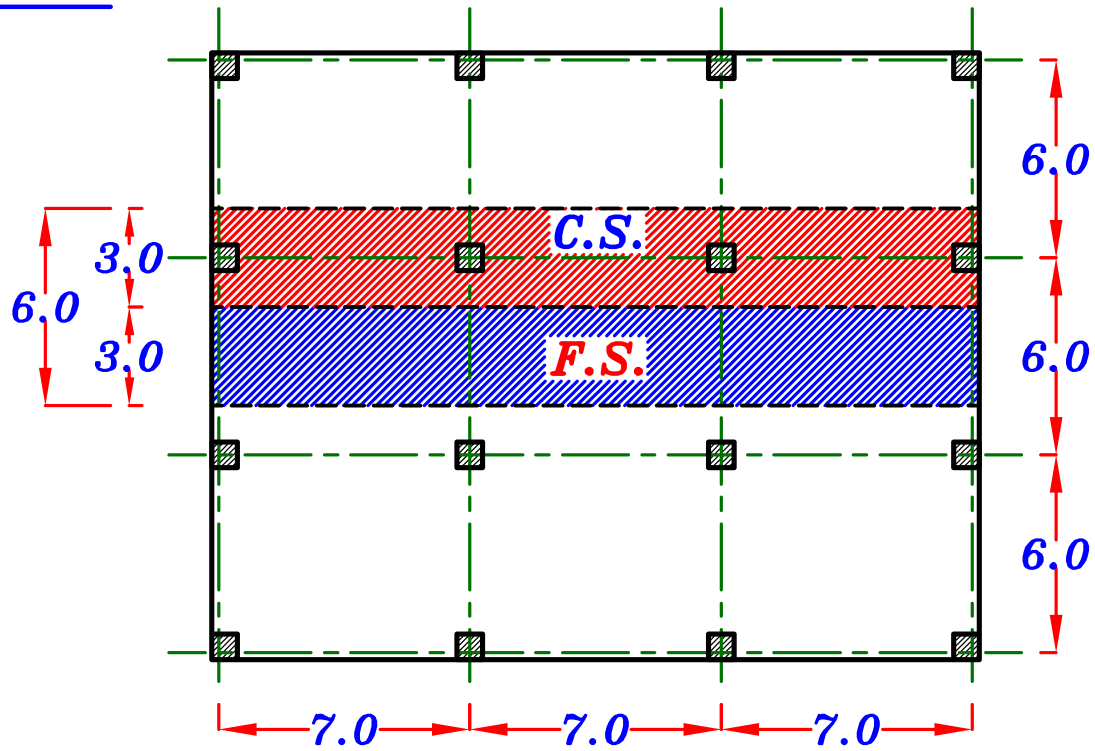
Moment in Short Direction.

$$M_o = \frac{(w_s * L_1)\left(L_2 - \frac{2}{3}D\right)^2}{8} = \frac{(18.30 * 7.0)\left(6.0 - \frac{2}{3} * 0.7\right)^2}{8}$$

$$\boxed{M_o = 490.26\ kN.m}$$ Short Direction

5- Distribute the B.M. (M_o) on C.S. & F.S.

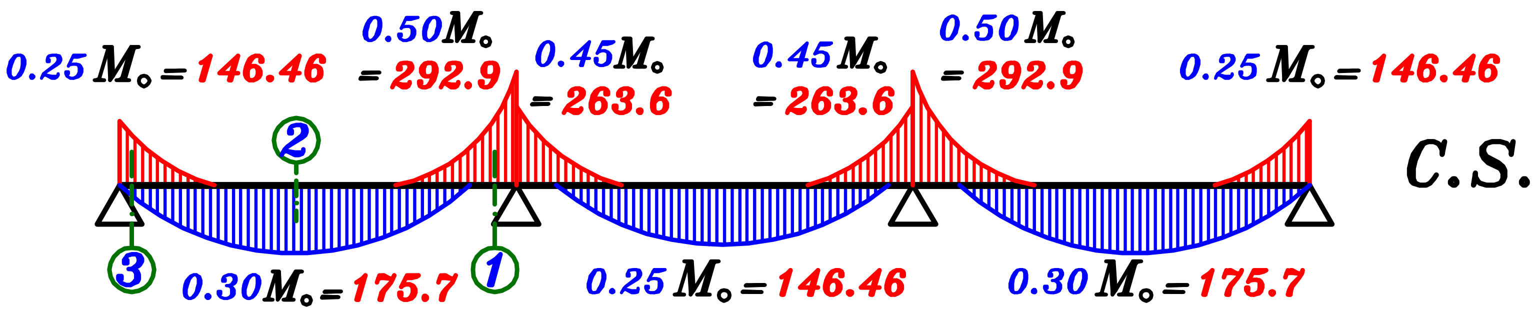
Long Direction.

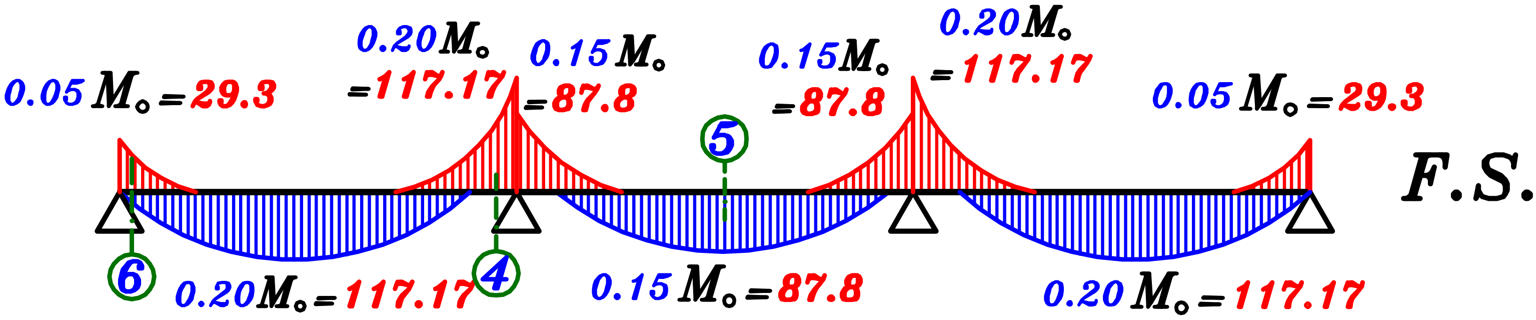


Column Strip width $= \dfrac{L_2}{2} = \dfrac{6.0}{2} = 3.0\ m$

Field Strip width $= 6.0 - 3.0 = 3.0\ m$

$\boxed{M_o = 585.84\ kN.m}$ **Long Direction**



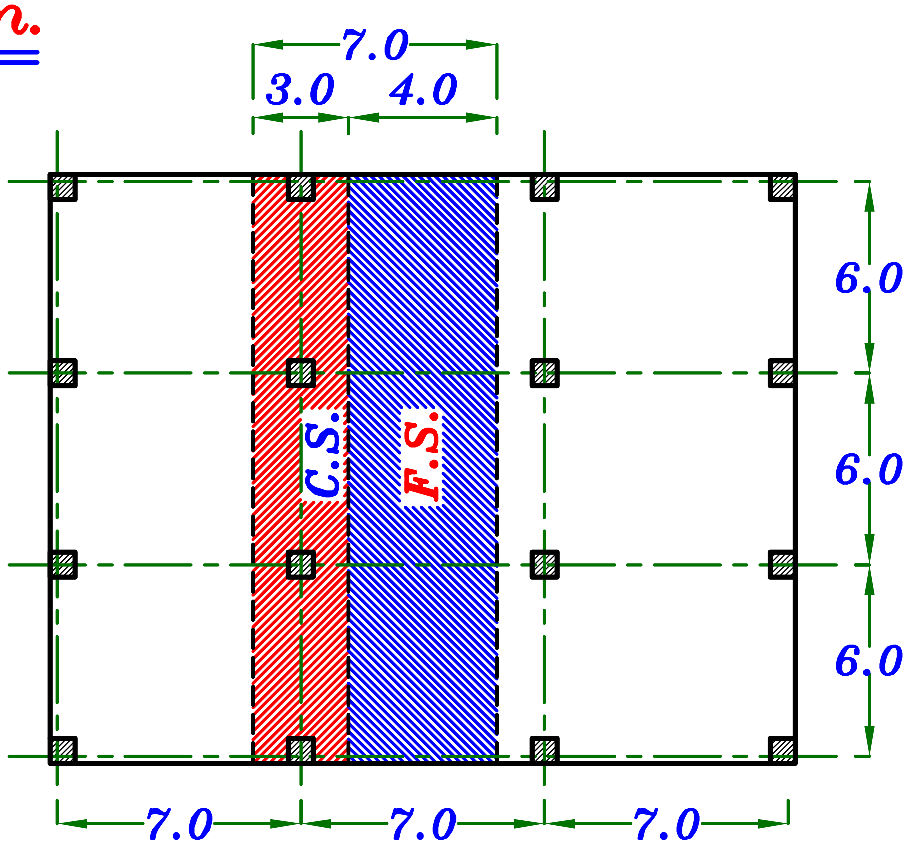


⑥ Design of sections.

$$d = t_s - 30\,mm = 220 - 30 = 190\ mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm²/b)	As (mm²/m)	No. of bars/m'
Column Strip	1	292.9	3000	190	3.04	0.747	5732	1910	8∅18\m
	2	175.7	3000	190	3.92	0.801	3206	1068	6∅16\m
	3	146.46	3000	190	4.30	0.814	2630	876	8∅12\m
Field Strip	4	117.17	3000	190	4.80	0.824	2078	692	7∅12\m
	5	87.8	3000	190	5.55	0.826	1554	518	5∅12\m
	6	29.3	3000	190	9.61	0.826	518	172	5∅12\m

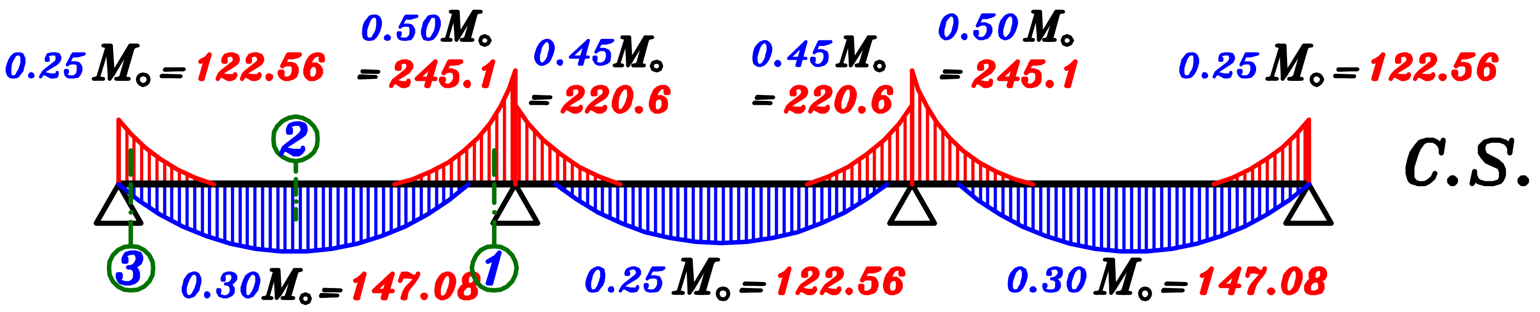
Short Direction.

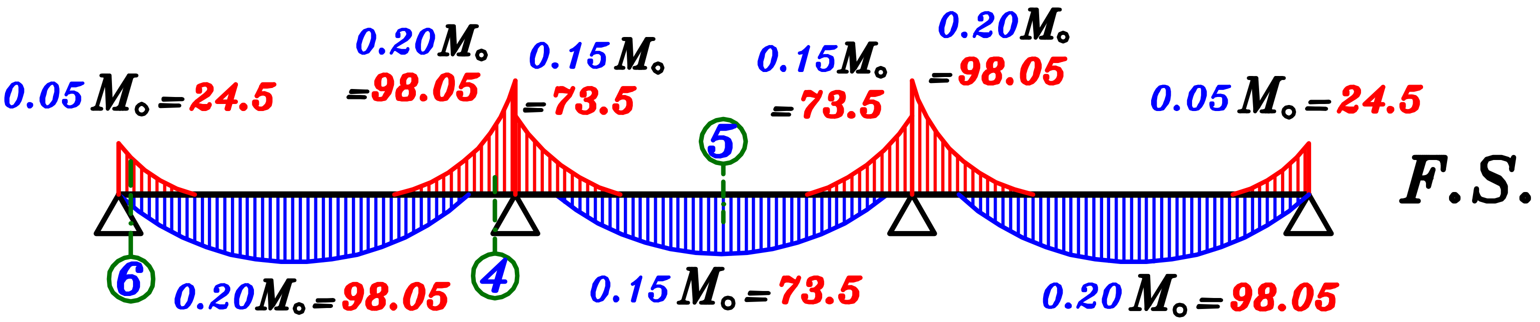


$$\text{Column Strip width} = \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

$$\text{Field Strip width} = L_1 - \frac{L_2}{2} = 7.0 - \frac{6.0}{2} = 4.0\ m$$

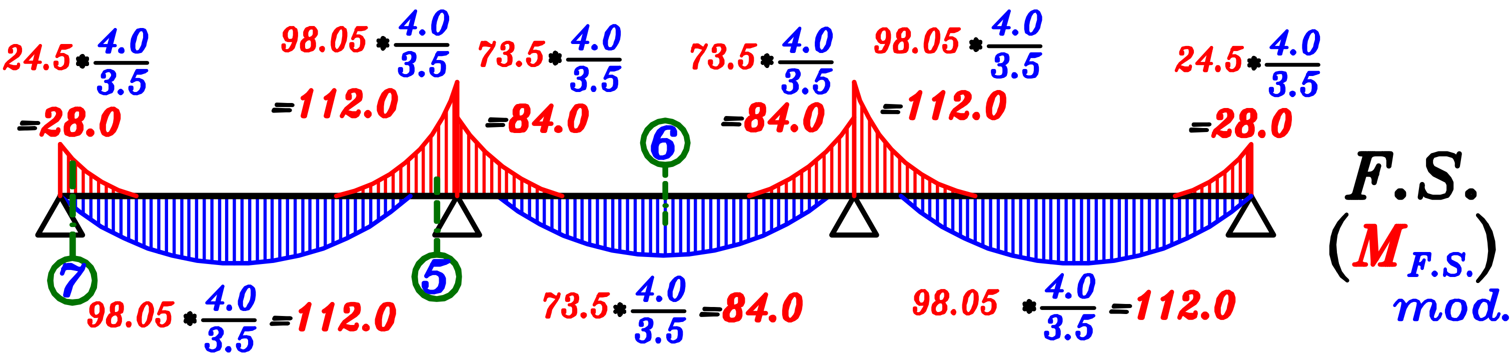
$$\boxed{M_o = 490.26\ kN.m}$$ Short Direction



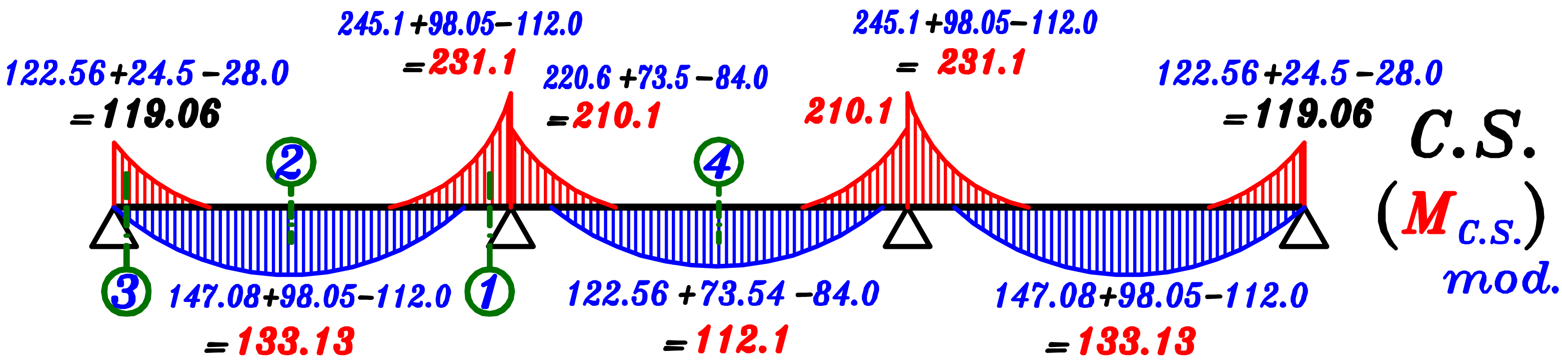


$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{4.0}{3.5}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.0}{3.5}}$$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

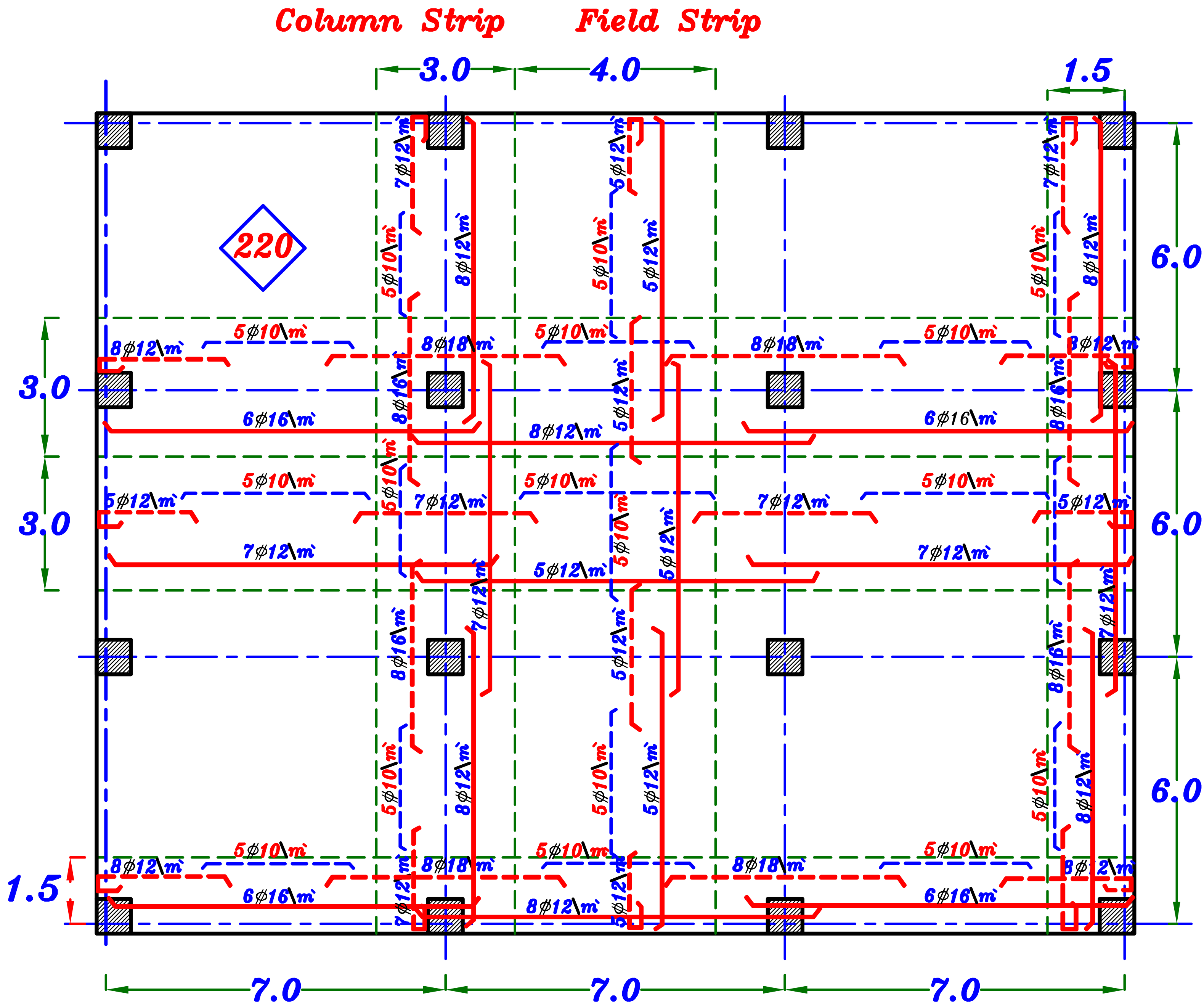


⑥ Design of sections.

$$d = t_s - 40\ mm = 220 - 40 = 180\ mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm^2/b)	As (mm^2/m)	No. of bars/m
Column Strip	1	231.1	3000	180	3.24	0.764	4668	1556	8ϕ16\m
	2	133.13	3000	180	4.27	0.812	2530	843	8ϕ12\m
	3	119.06	3000	180	4.52	0.819	2242	747	7ϕ12\m
	4	112.1	3000	180	4.65	0.821	2107	702	7ϕ12\m
Field Strip	5	112.0	4000	180	5.37	0.826	2093	523	5ϕ12\m
	6	84.0	4000	180	6.21	0.826	1569	392	5ϕ12\m
	7	28.0	4000	180	10.75	0.826	523	130	5ϕ12\m

7- Details of RFT.

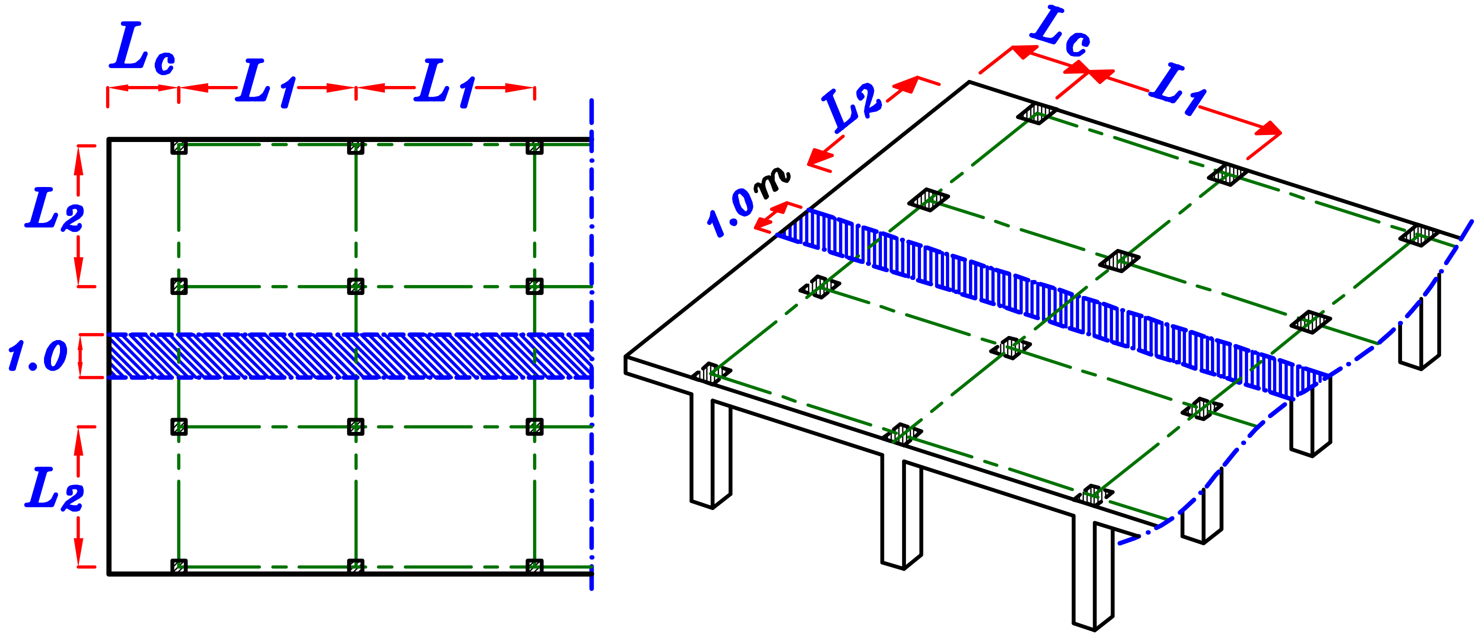


الـ Column Strip التى فى الطرف يكون تسليحها فى المتر هو نفس تسليح الـ Column Strip الرئيسيه .

Cantilever Flat Slab.

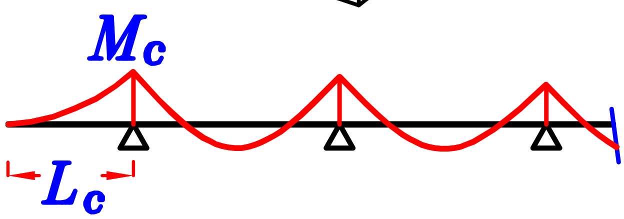
لحساب العزم على الـ Cantilever Flat Slab

نأخذ شريحه فى اتجاه الـ Cantilever عرضها -١,٠ م و نحسب العزم لها .

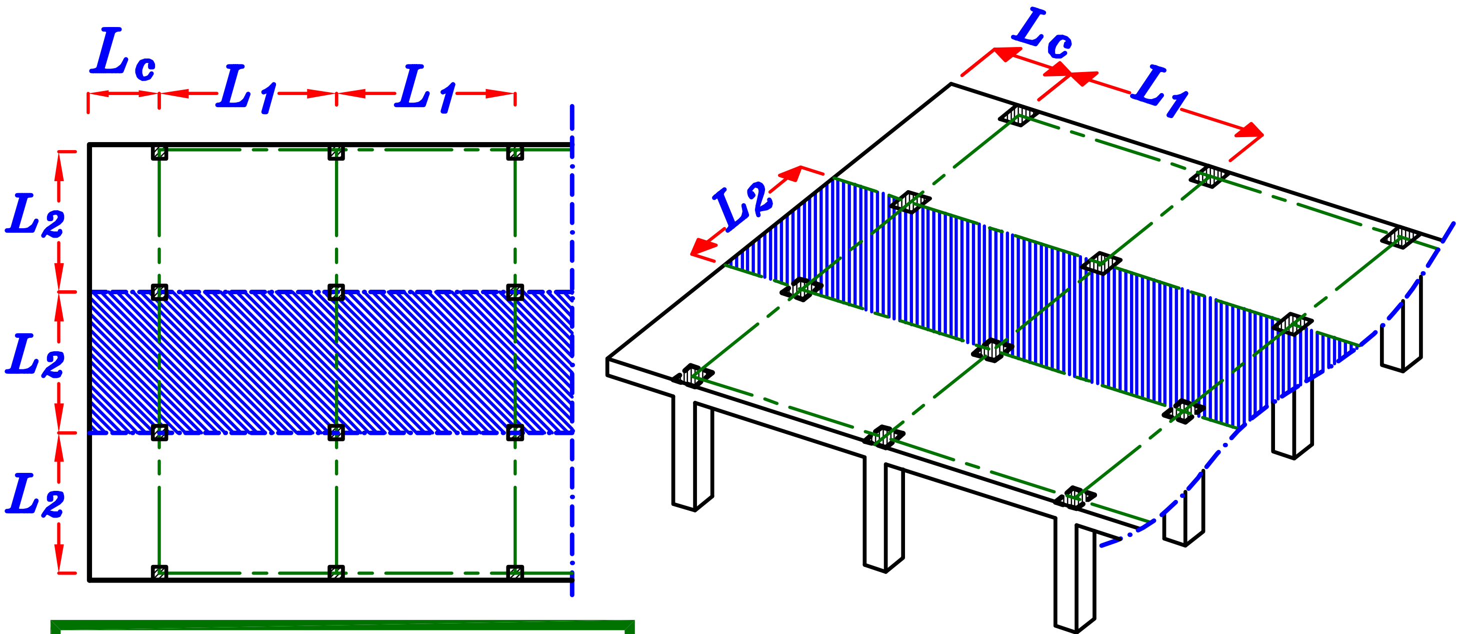


$$M_{Cant.} = \frac{w_s * (L_c)^2}{2}$$

عزم شريحه عرضها -١,٠ م

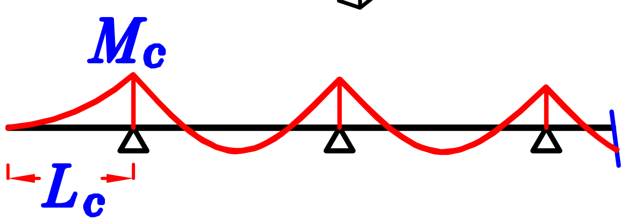


ثم نحسب عزم الـ Cantilever للشريحه كلها التى عرضها L_2



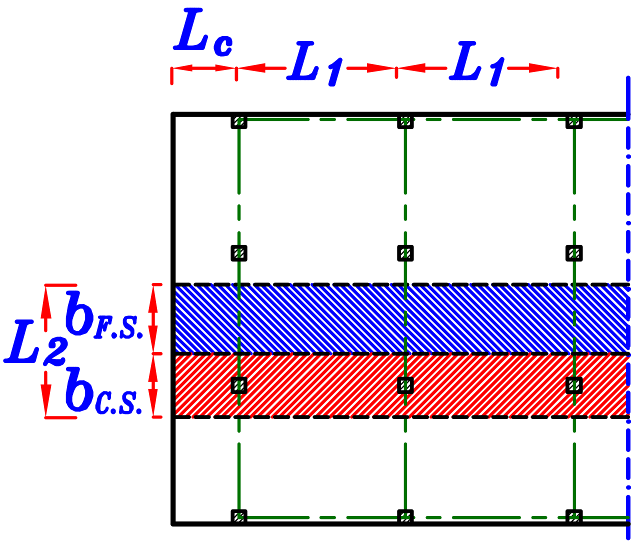
$$M_{Cant.} = \frac{w_s * (L_c)^2}{2} * L_2$$

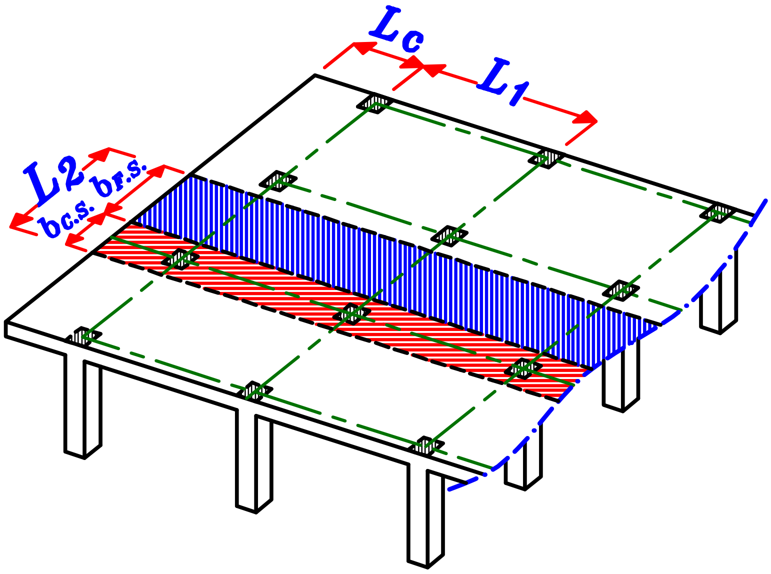
عزم شريحه عرضها من C.L. الى C.L.



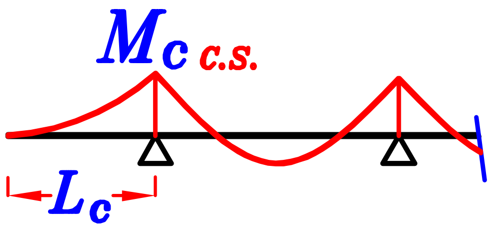
يتم توزيع عزم شريحه الباكيه كلها من C.L. الى C.L.

على ال Column Strip و ال Field Strip كلا حسب عرضه الحقيقى

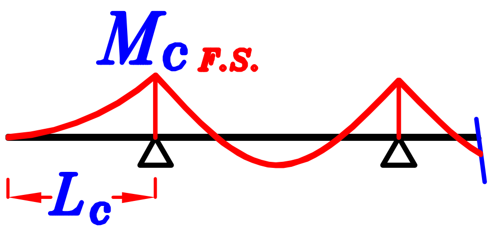




$$M_{Cant.(C.S.)} = \frac{w_s * (L_c)^2}{2} * (b_{C.S.})$$



$$M_{Cant.(F.S.)} = \frac{w_s * (L_c)^2}{2} * (b_{F.S.})$$



حيث $(b_{C.S.})$ هو العرض الحقيقى لل Column Strip

و ال $(b_{F.S.})$ هو العرض الحقيقى لل Field Strip

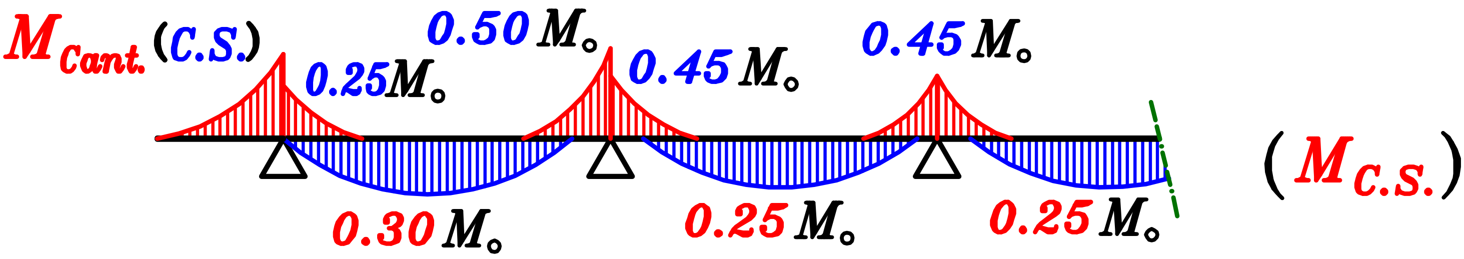
فى الكود القديم كان يؤخذ قيمه :

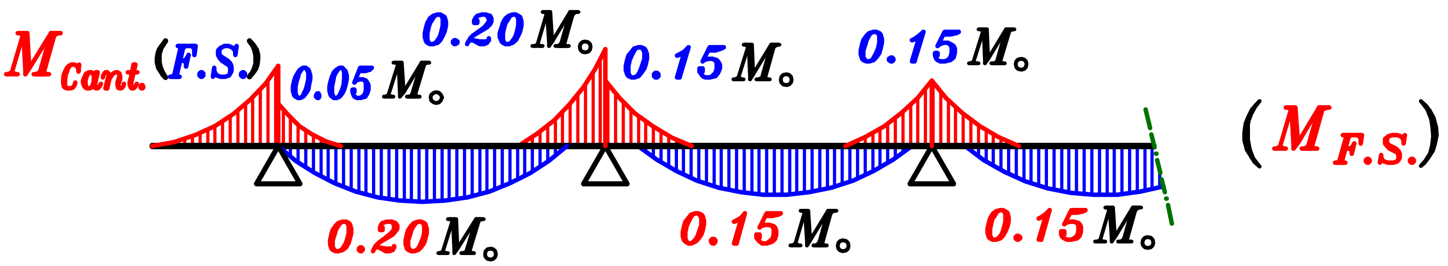
لن نستخدم هذه القوانين فى هذا الملف

$$M_{Cant.(C.S.)} = 0.75 * \frac{w_s * (L_c)^2}{2} * L_2$$

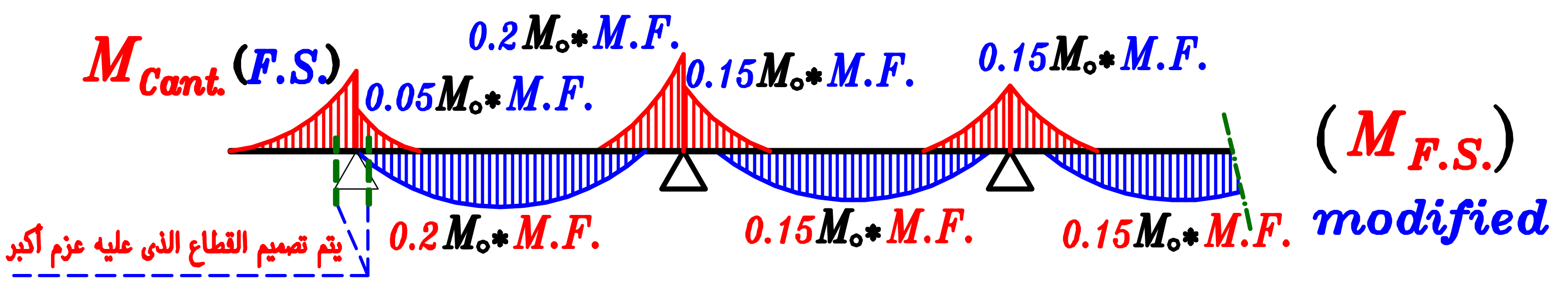
$$M_{Cant.(F.S.)} = \text{الاكبر} \begin{bmatrix} \frac{w_s * (L_c)^2}{2} * (b_{F.S.}) \\ 0.25 * \frac{w_s * (L_c)^2}{2} * L_2 \end{bmatrix}$$

لا يتم ضرب عزوم ال Cantilever فى modification Factor

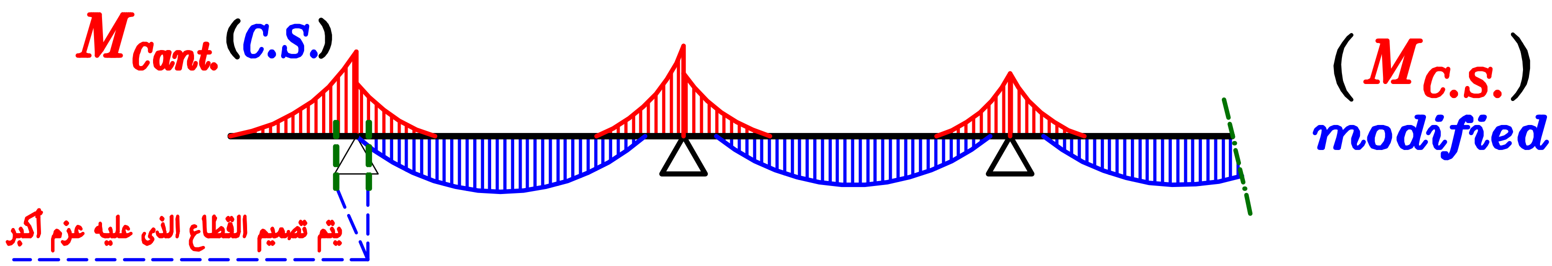




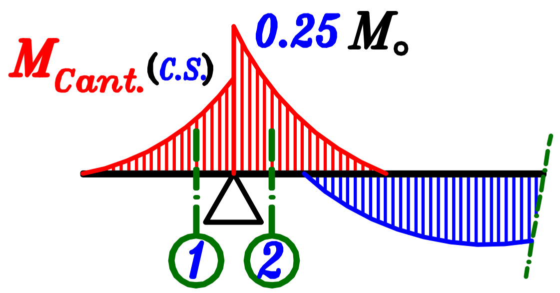
$$(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor}$$



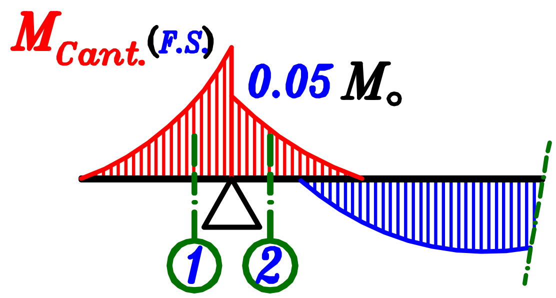
$$(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}$$



Design.



Column Strip

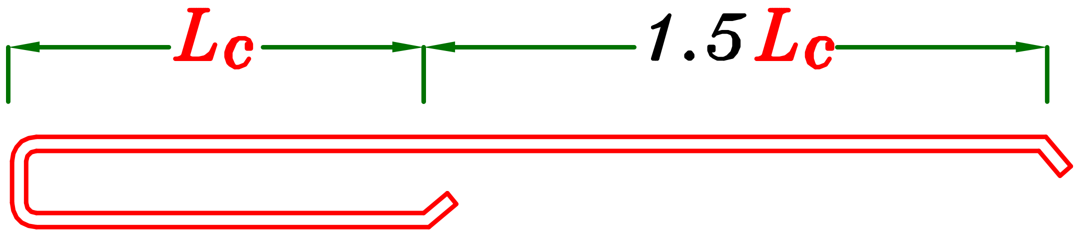


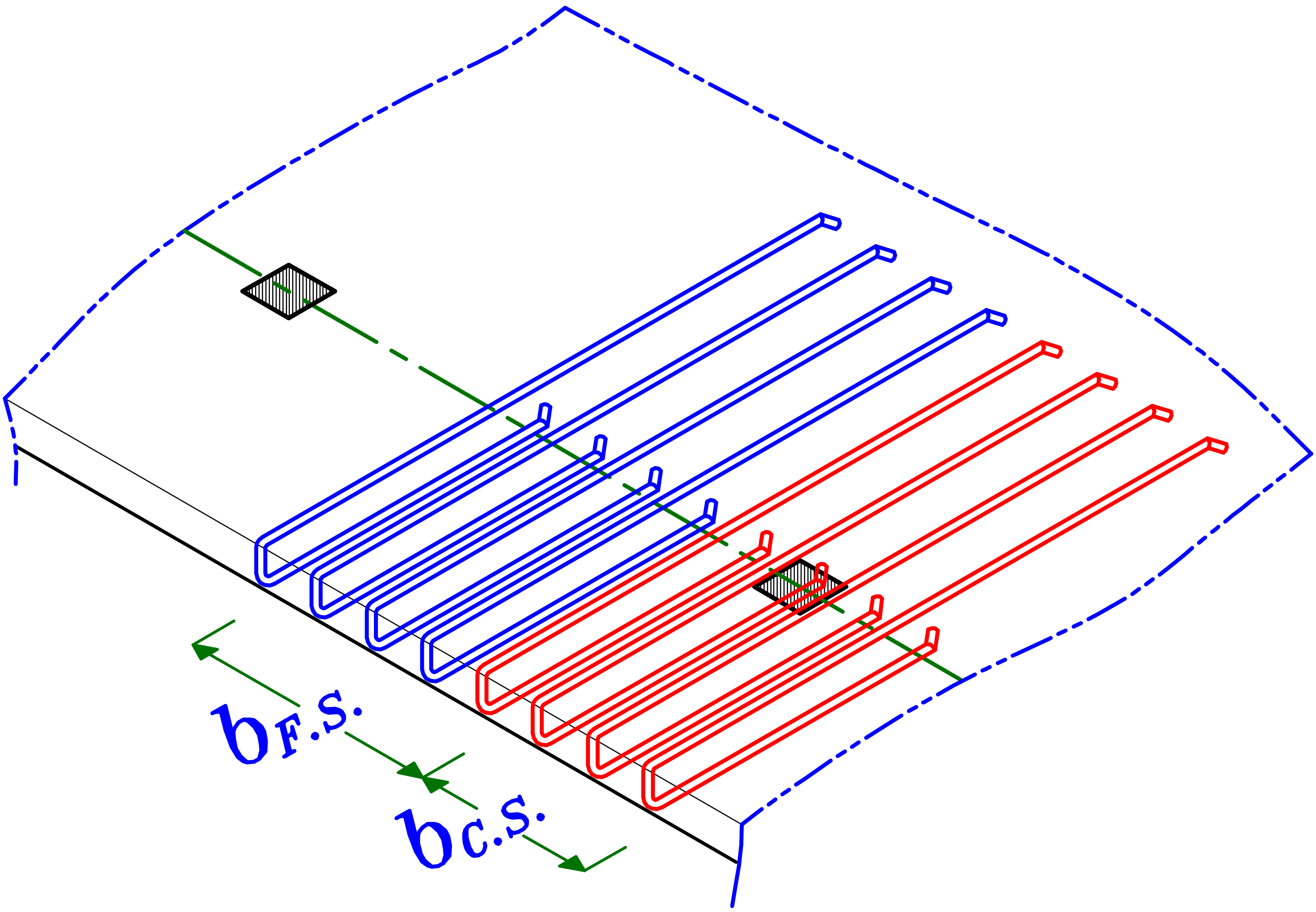
Field Strip

نصمم القطاع الذى عليه عزم أكبر من ① أو ②

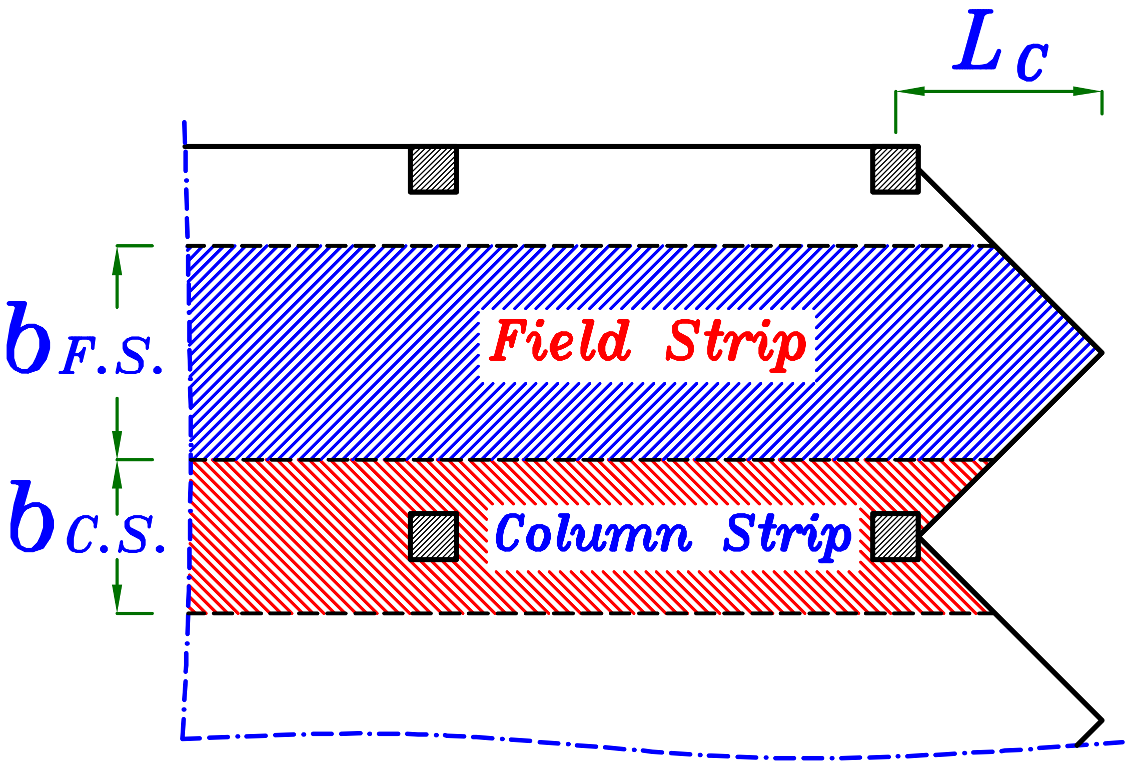
نصمم القطاع الذى علية عزم أكبر من ① أو ②

و يكون التسليح عباره عن شوكه .



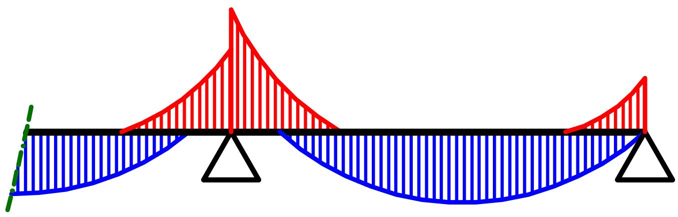


Case of Cantilever with variable length.



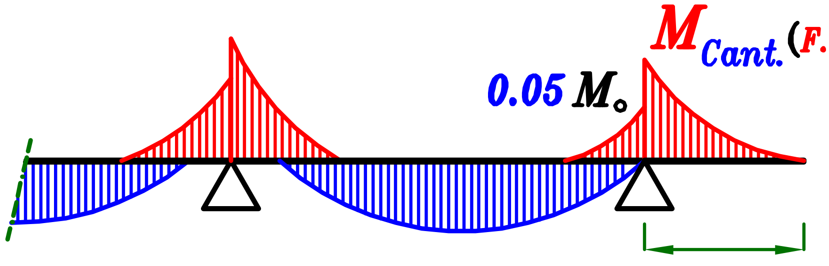
في هذه الحاله ممكن اهمال تأثير الكابولى على ال **Column Strip**

Column Strip

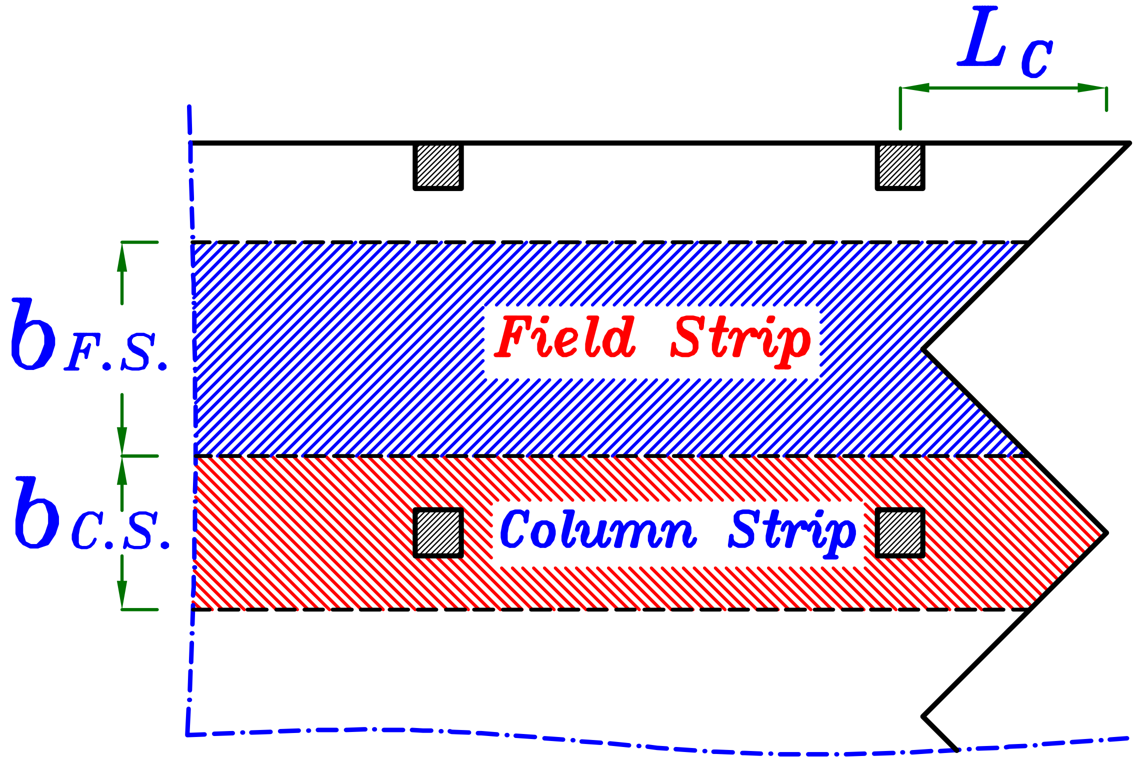


نأخذ تأثير ال **Cantilever** بطوله كله على ال **Field Strip**

Field Strip

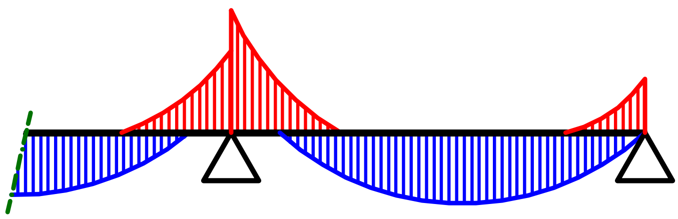


$$M_{Cant.(F.S.)} = \frac{w_s * (L_c)^2}{2} * (b_{F.S.})$$



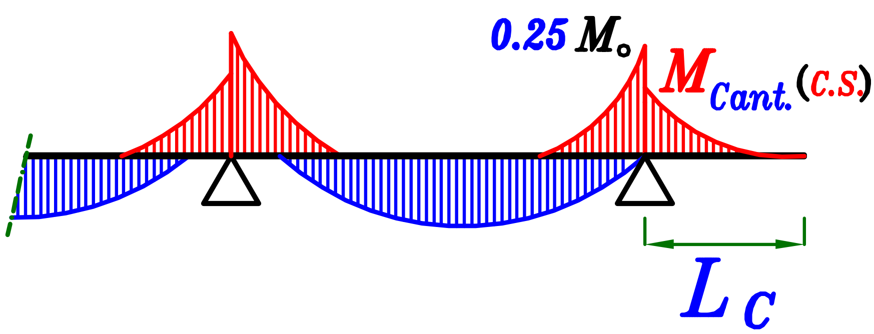
فى هذه الحاله ممكن اهمال تأثير الكابولى على الـ Field Strip

Field Strip



نأخذ تأثير الـ Cantilever بطوله كله على الـ Column Strip

Column Strip

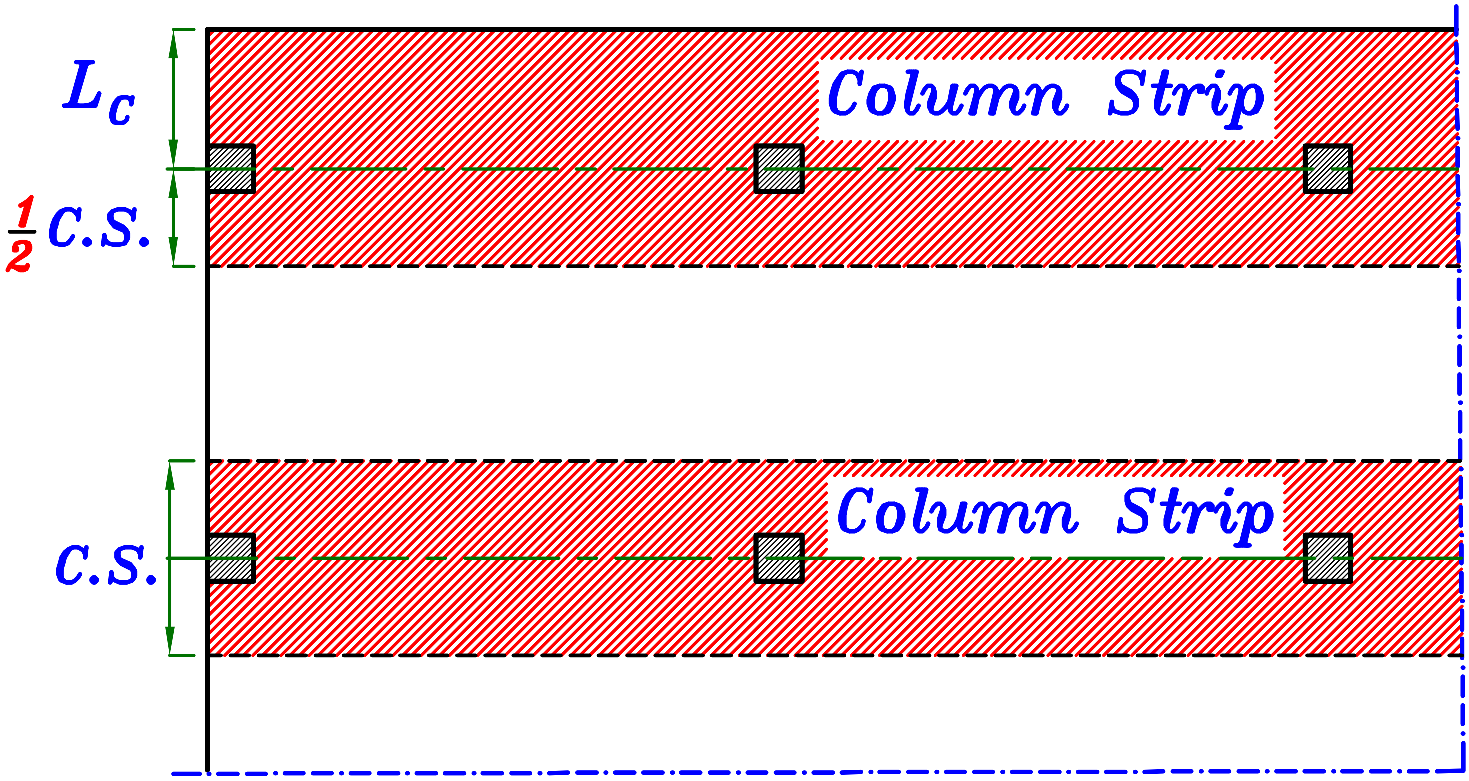


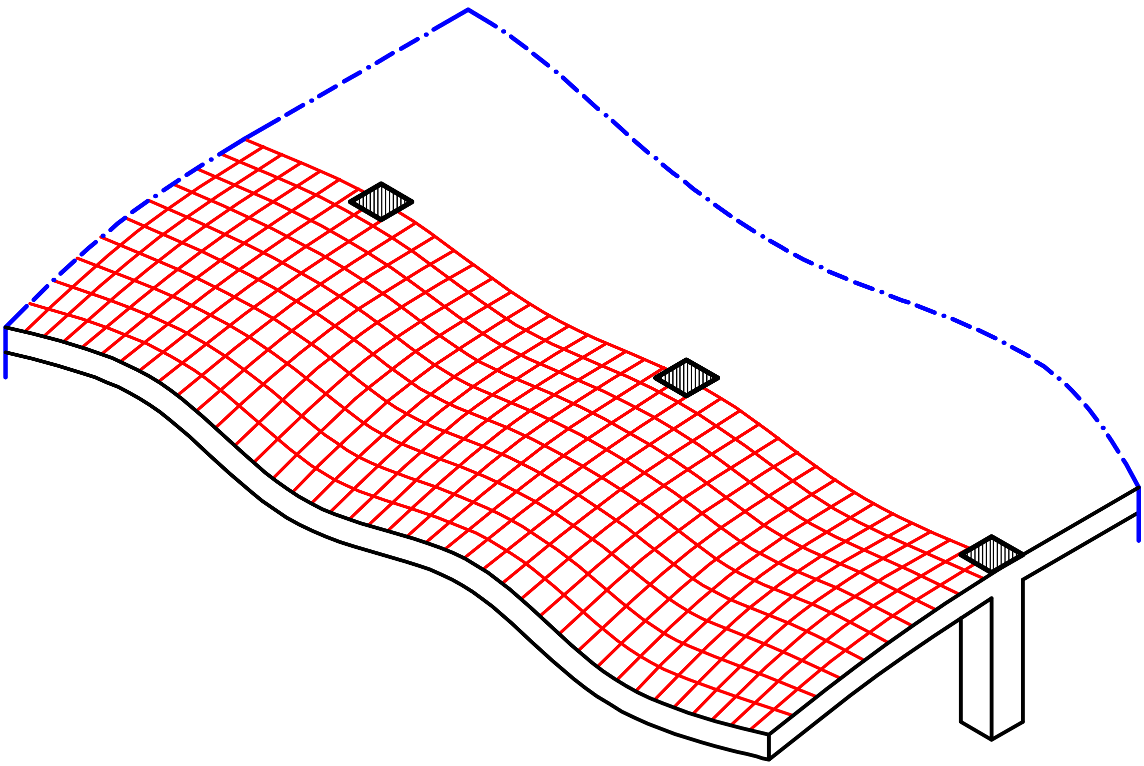
$$M_{Cant.(c.s.)} = \frac{w_s * (L_c)^2}{2} * (b_{c.s.})$$

توجد حالتان لوضع الحديد العمودى على ال Cantilever

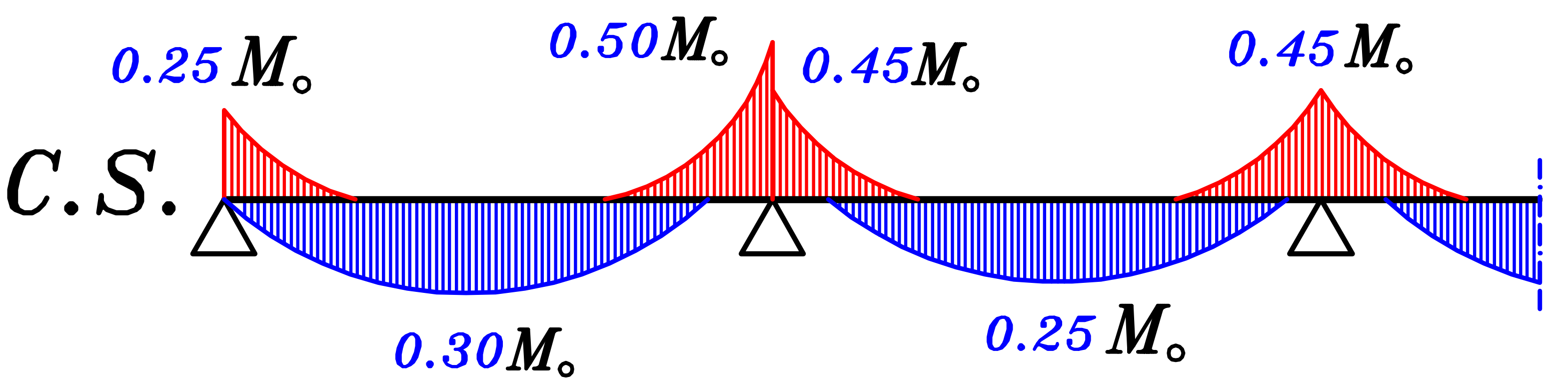
١- لا يوجد marginal Beam

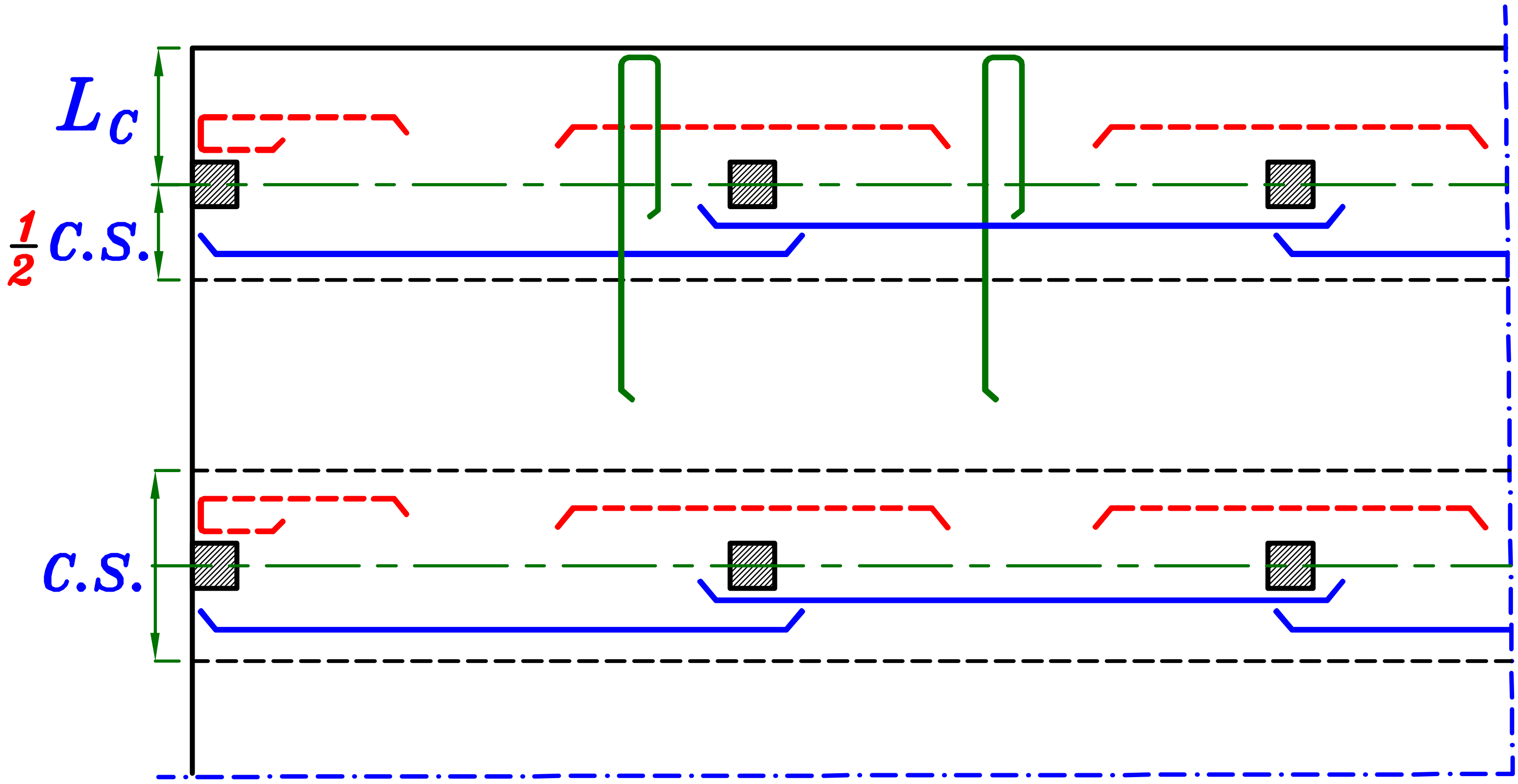
اذا لم توجد marginal Beam نأخذ شريحه عموديه على ال Cantilever





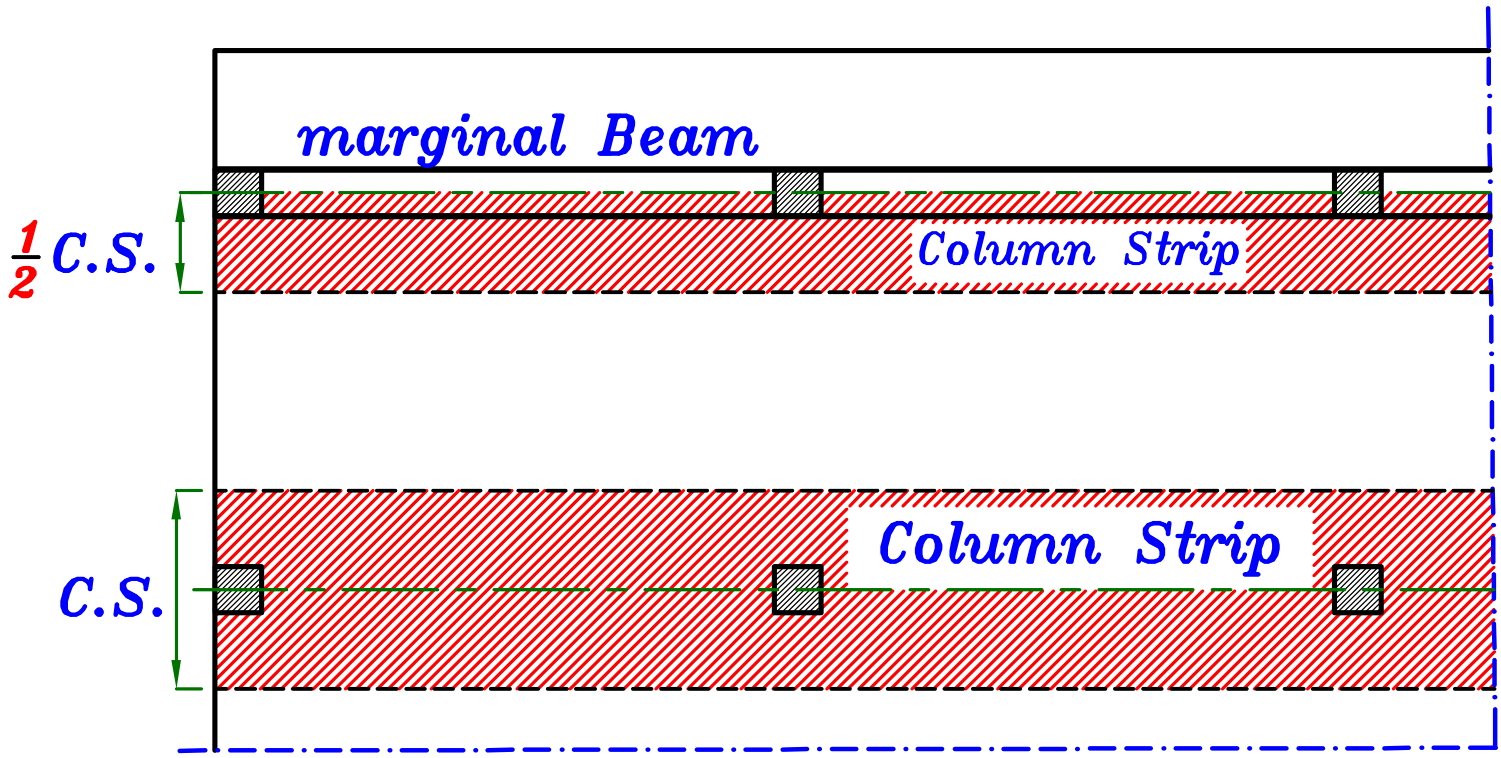
يتم تسليح الشريحه العموديه على الـ *Cantilever*
بنفس تسليح الـ *Column Strip*

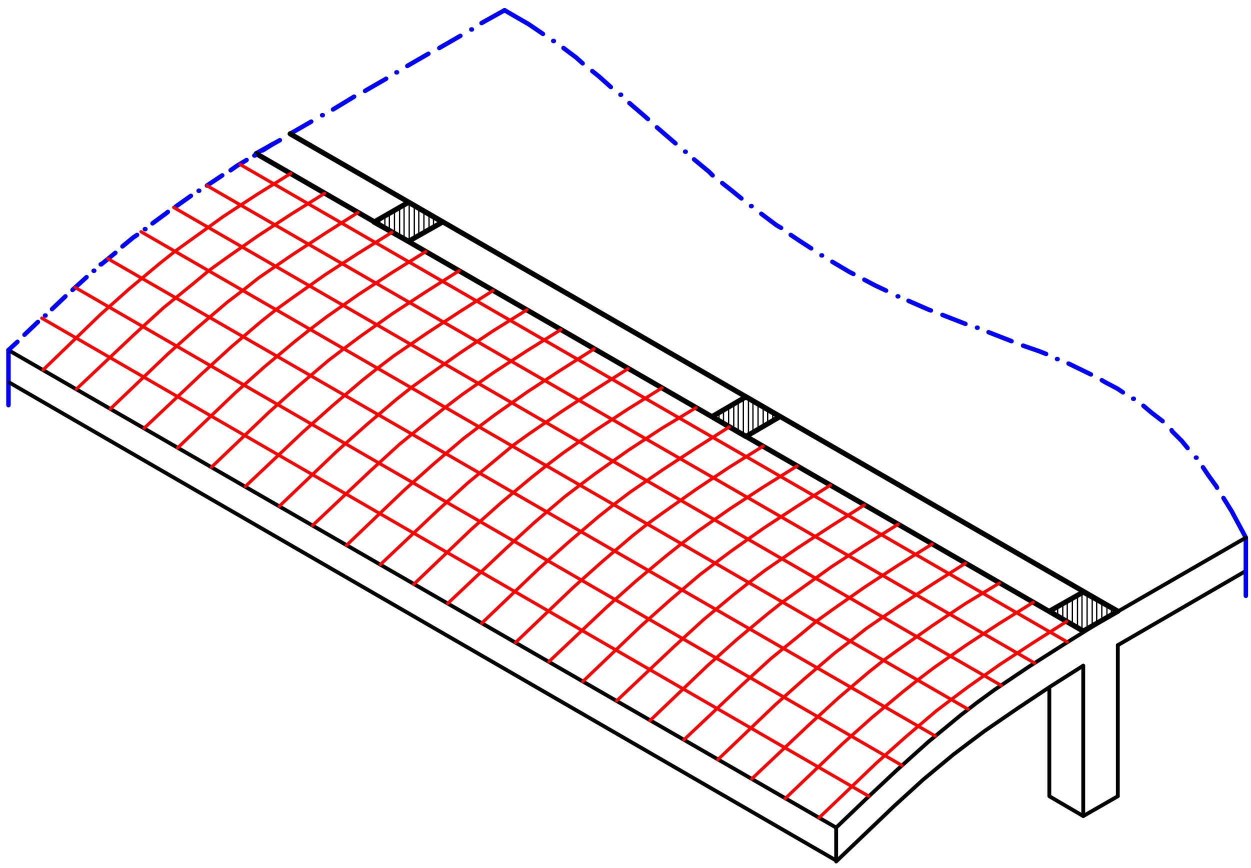




٢- يوجد marginal Beam

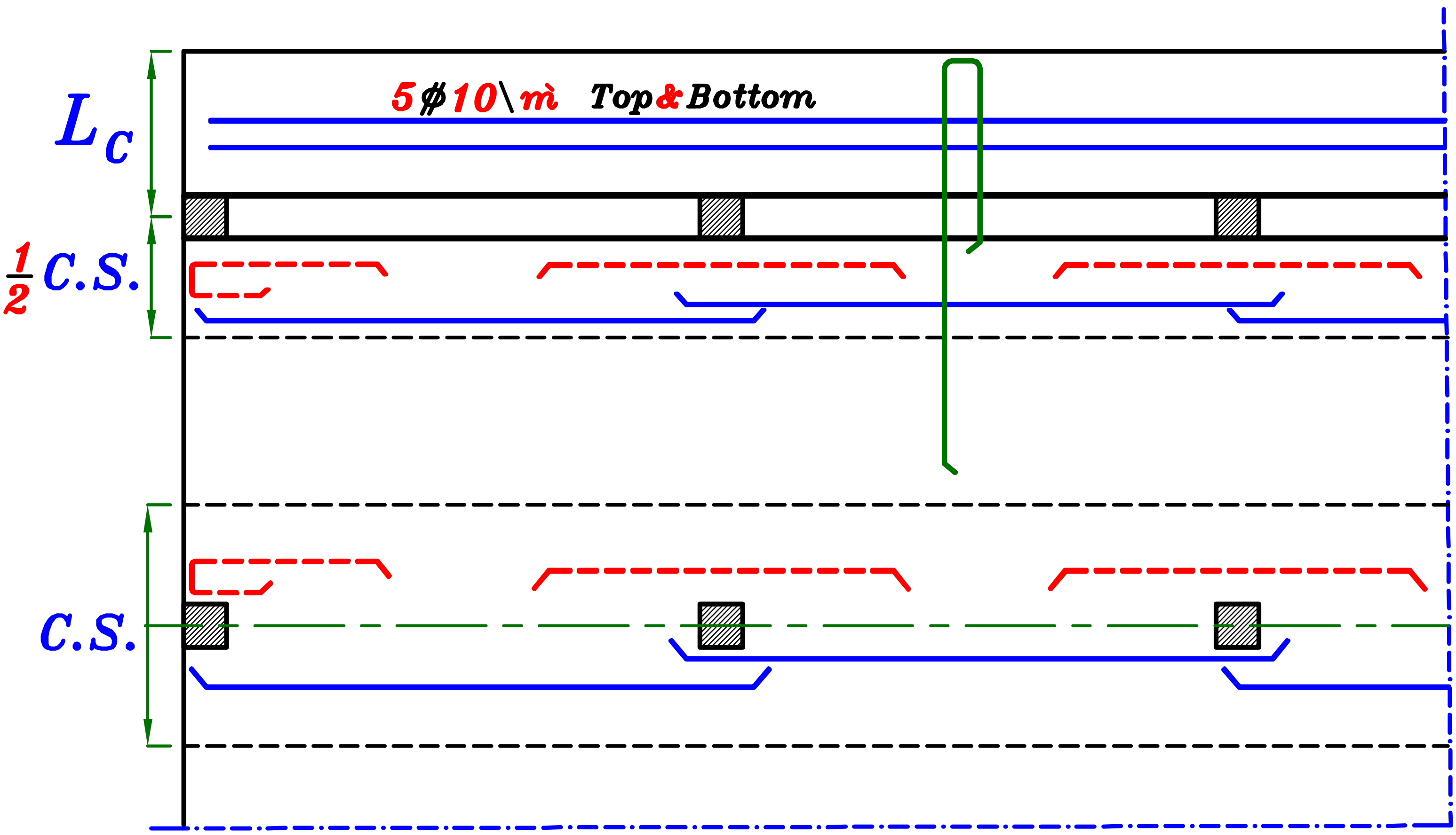
اذا وجدت marginal Beam يعتبر Cantilever Solid Slab





اذا وجدت **Beam marginal** يعتبر **Cantilever Solid Slab**

و نضع تسليح عمودى على الـ **Cantilever** عباره عن **5ϕ10\m̀ Top & Bottom**



Example.

The given plan shows general layout of a Flat slab Floor
The column height 3.50 m

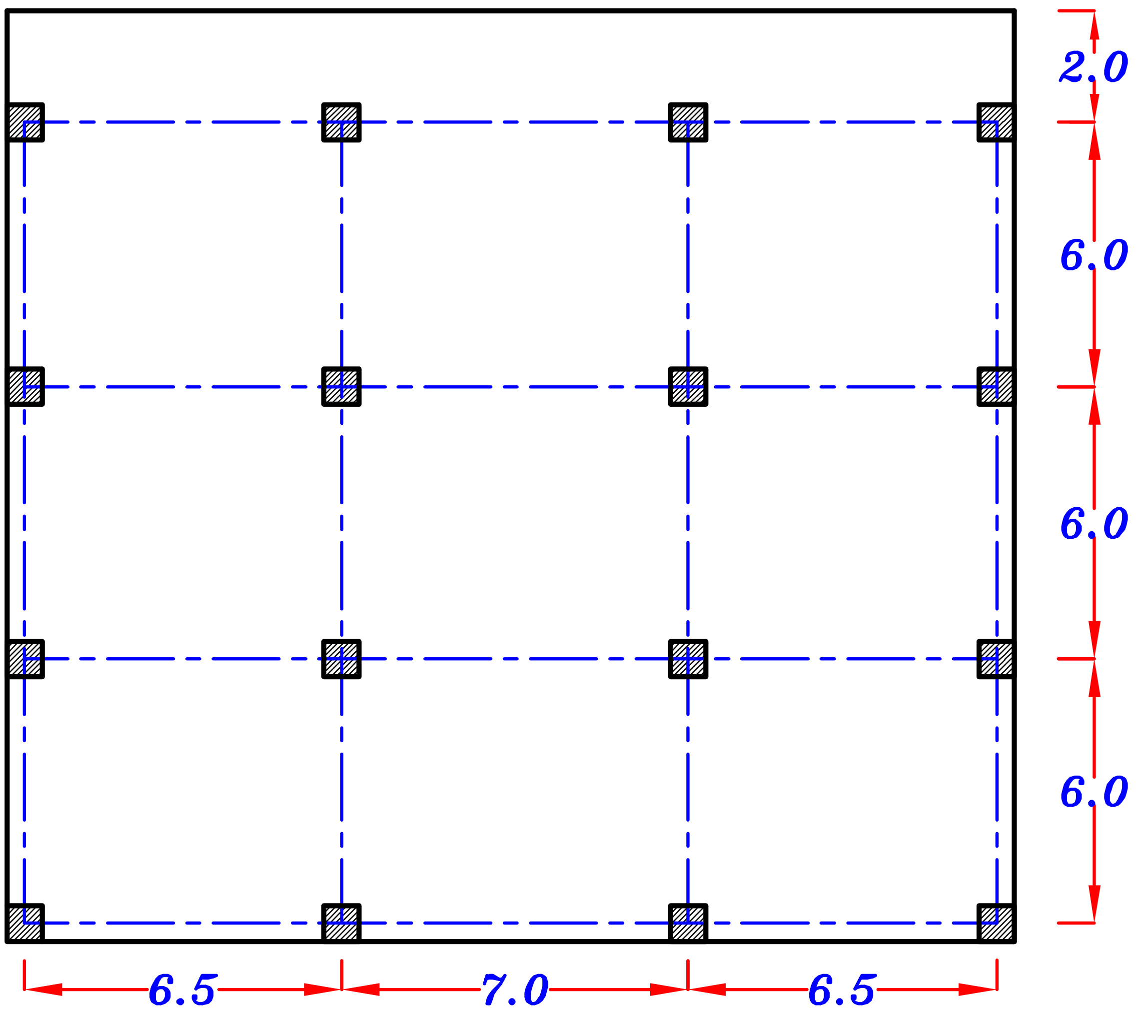
Data.

$F_{cu} = 30\ N\backslash mm^2$, $F_y = 400\ N\backslash mm^2$

$F.C. = 2.0\ kN\backslash m^2$, $L.L. = 3.0\ kN\backslash m^2$, $Walls = 2.0\ kN\backslash m^2$

Req.

1. Check punching on columns
2. Using empirical method calculate the moments For both the Field strip and the column strip in both directions.
3. Design the sections of the slab. and draw details of reinforcement in plan.



Solution.

1- Concrete Dimensions.

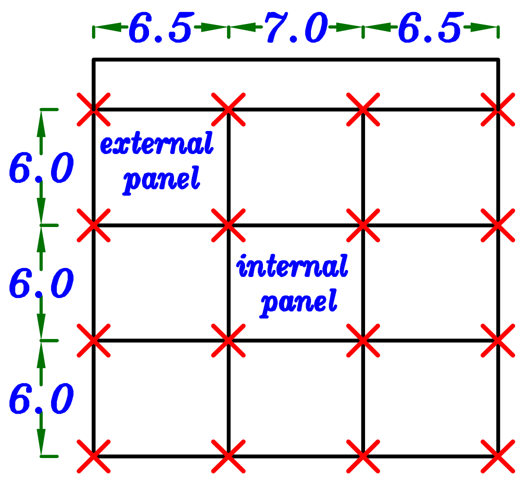
Column dimensions.

$$b_{Col.} = \begin{cases} 300 \text{ mm} \\ \dfrac{H}{15} = \dfrac{3500}{15} = 233.3 \text{ mm} \\ \dfrac{L_1}{20} = \dfrac{7000}{20} = 350 \text{ mm} \end{cases}$$

$$\boxed{b_{Col.} = 350 \text{ mm}}$$

$(350 * 350)$

Slab Thikness.



$L_1 = 6.5$ m External Panel

$L_1 = 7.0$ m Internal Panel

External panel $t_s = \dfrac{L_1}{32} = \dfrac{6500}{32} = 203.1$ mm

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7000}{36} = 194.4$ mm

$$\boxed{t_s = 220 \text{ mm}}$$

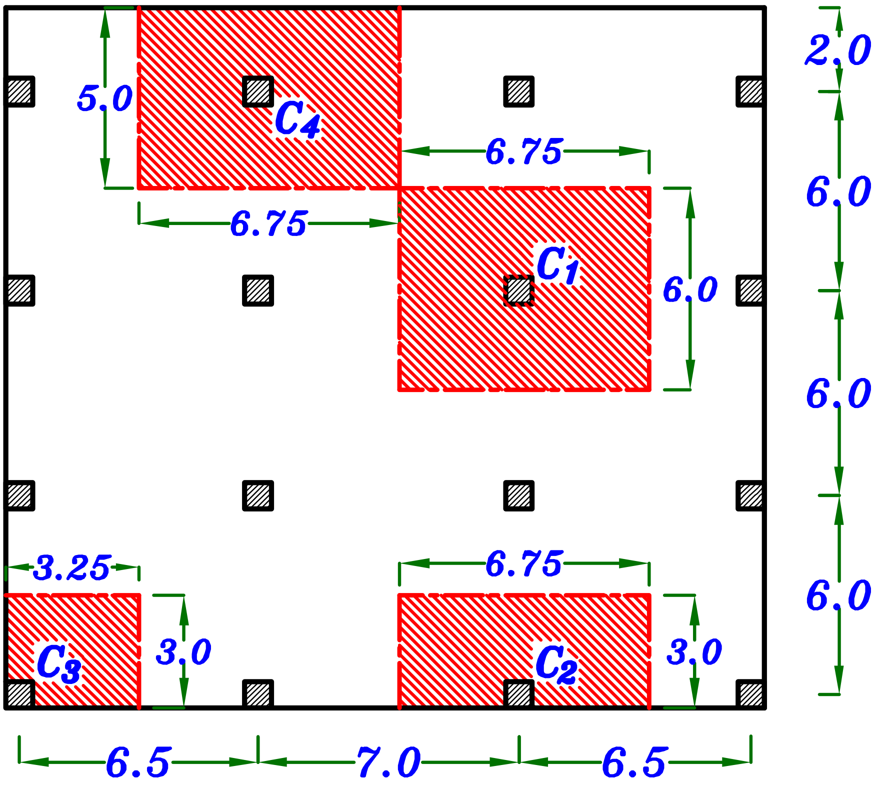
2- Loads on the Slab.

$$w_{s\,U.L.} = 1.4\,(t_s\,\gamma_c + F.C. + Walls) + 1.6\,(L.L.)$$

$$w_{s\,U.L.} = 1.4\,(0.22 * 25 + 2.0 + 2.0) + 1.6\,(3.0) = 18.10 \ kN\backslash m^2$$

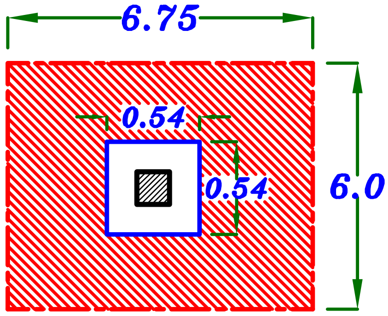
3 – Check Punching.

كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى



ما عدا العمود
بجوار الـ Cantilever
يحمل الـ Cantilever كله

C_1 Interior Column.



$$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$$

$$C + d = 0.35 + 0.19 = 0.54\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 18.10\left[6.0 * 6.75 - 0.54 * 0.54\right] = 727.7\ kN$$

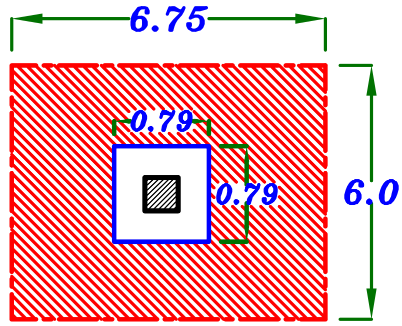
$$A_p = (b_o * d) = (4 * 540) * 190 = 410400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{727.7 * 10^3}{410400} * 1.15 = 2.04\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$$

$$\boxed{q_{pu} > q_{pcu}}$$ $\longrightarrow$ Unsafe punching
Increase dimensions of the column

C_1 Interior Column.



Take the Column (600 * 600)

$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$

$C+d = 0.60 + 0.19 = 0.79\ m$

$Q_{pu} = w_s\ [L_1 * L_2 - (C_1+d)(C_2+d)]$

$Q_{pu} = 18.10\ [6.0 * 6.75 - 0.79 * 0.79] = 721.7\ kN$

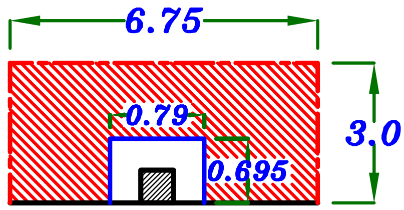
$A_p = (b_o * d) = (4 * 790) * 190 = 600400\ mm^2$

$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{721.7 * 10^3}{600400} * 1.15 = 1.38\ N/mm^2$

$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$

$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.

$C2$ Edge Column.



Take the Column (600 * 600)

$C+d = 0.60 + 0.19 = 0.79\ m$

$C+\frac{d}{2} = 0.60 + \frac{0.19}{2} = 0.695\ m$

$Q_{pu} = w_s\ [L_1 * L_2 - (C_1+d)(C_2+d)]$

$Q_{pu} = 18.10\ [6.75 * 3.0 - 0.79 * 0.695] = 356.6\ kN$

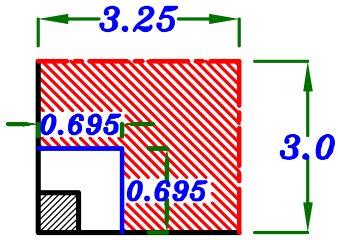
$A_p = (b_o * d) = (790 + 2 * 695) * 190 = 414200\ mm^2$

$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{356.6 * 10^3}{414200} * 1.30 = 1.12\ N/mm^2$

$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$

$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.

C_3 Corner Column.



Take the Column (600 * 600)

$$C+\frac{d}{2} = 0.60 + \frac{0.19}{2} = 0.695 \ m$$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 18.10 \left[3.25 * 3.0 - 0.695 * 0.695\right] = 167.7 \ kN$$

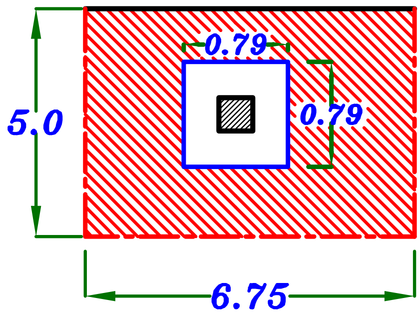
$$A_p = (b_o * d) = (2 * 695) * 190 = 264100 \ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{167.7 * 10^3}{264100} * 1.50 = 0.952 \ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41 \ N/mm^2$$

$$\boxed{q_{pu} < q_{pcu}} \longrightarrow \text{Safe Punching.}$$

C_4 Column at Cantilever.



Take the Column (600 * 600)

$$C+d = 0.60 + 0.19 = 0.79 \ m$$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 18.10 \left[5.0 * 6.75 - 0.79 * 0.79\right] = 599.6 \ kN$$

$$A_p = (b_o * d) = (4 * 790) * 190 = 600400 \ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{599.6 * 10^3}{600400} * 1.15 = 1.148 \ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41 \ N/mm^2$$

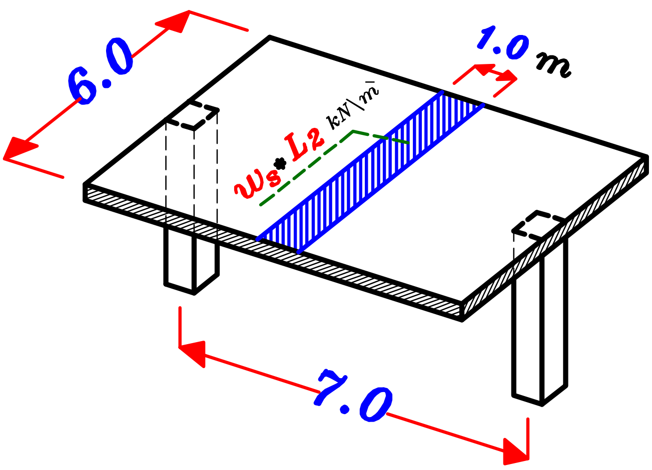
$$\boxed{q_{pu} < q_{pcu}} \longrightarrow \text{Safe Punching.}$$

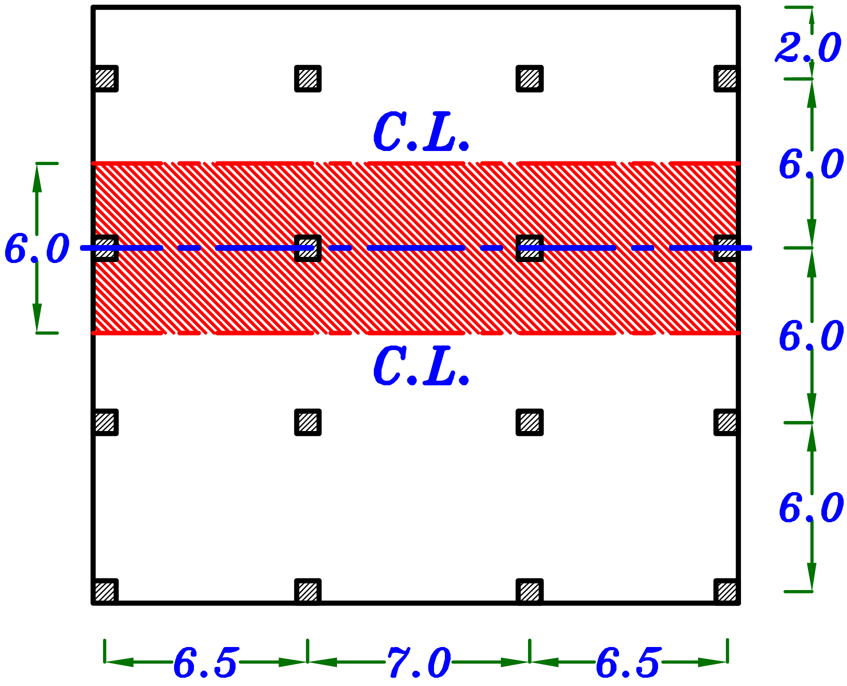
4-Take a Strips in the slabs at the long and short directions.
The strip width From C.L. the slab to C.L. the slab.
and Calculate the moment on the panel.

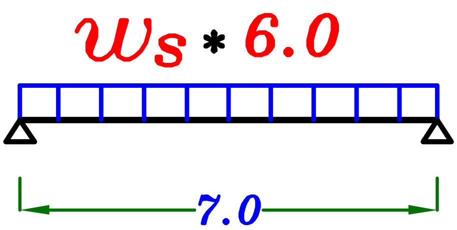
Strip at Long Direction.

Span $= 7.0\ m$

Width $= 6.0\ m$







Moment in Long Direction.

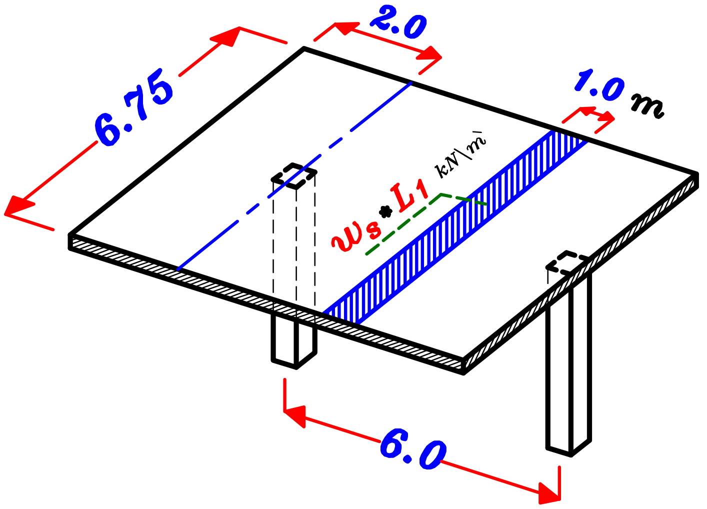
$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8} = \frac{(18.10 * 6.0)\left(7.0 - \frac{2}{3} * 0.6\right)^2}{8}$$

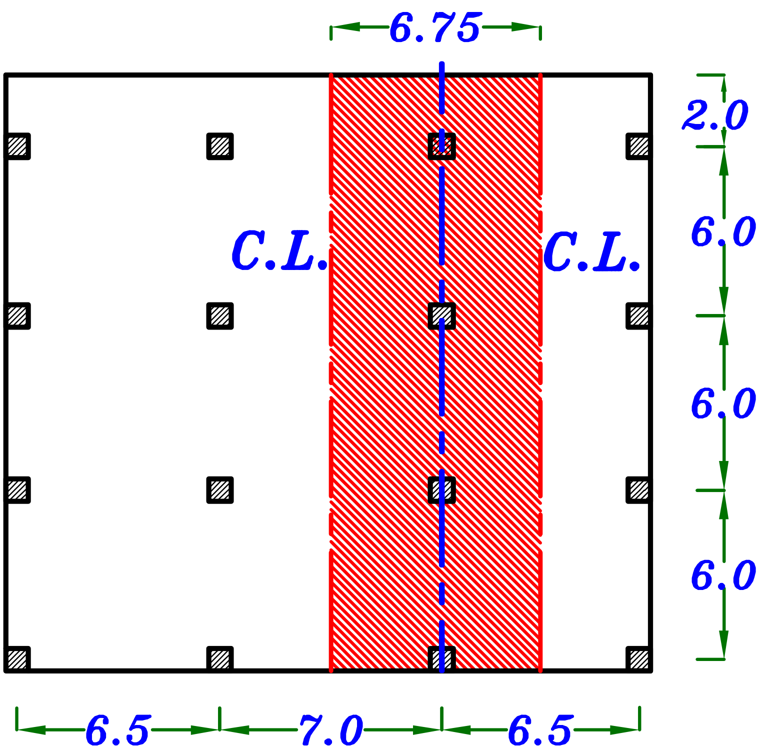
$$\boxed{M_o = 591.3\ kN.m}$$ Long Direction

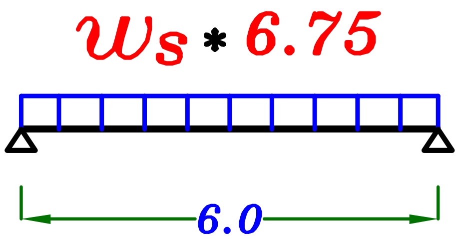
Strip at Short Direction.

Span $= 6.0$ m

Width $= 6.75$ m







Moment in Short Direction.

$$M_o = \frac{(w_s * L_1)\left(L_2 - \frac{2}{3}D\right)^2}{8} = \frac{(18.10 * 6.75)\left(6.0 - \frac{2}{3} * 0.6\right)^2}{8}$$

$$\boxed{M_o = 478.9 \text{ kN.m}}$$ Short Direction.

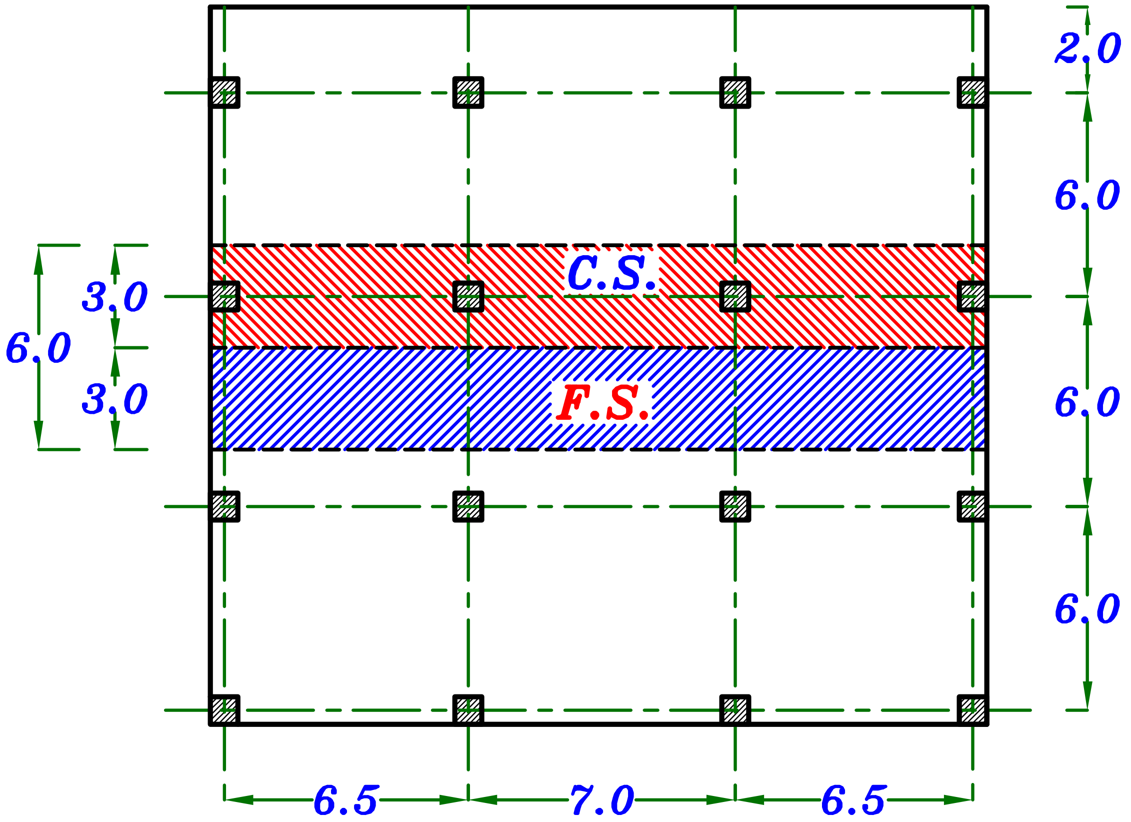
Cantilever Moment.

$$M_{Cant.}\backslash m` = \frac{w_s * (L_c)^2}{2} = \frac{18.10 * (2.0)^2}{2} = 36.2 \text{ kN.m/m}$$

$$M_{Cant.}(C.S.) = M_{Cant.}\backslash m` * b_{C.S.} = 36.2 * 3.0 = 108.6 \text{ kN.m}$$

$$M_{Cant.}(F.S.) = M_{Cant.}\backslash m` * b_{F.S.} = 36.2 * 3.75 = 135.75 \text{ kN.m}$$

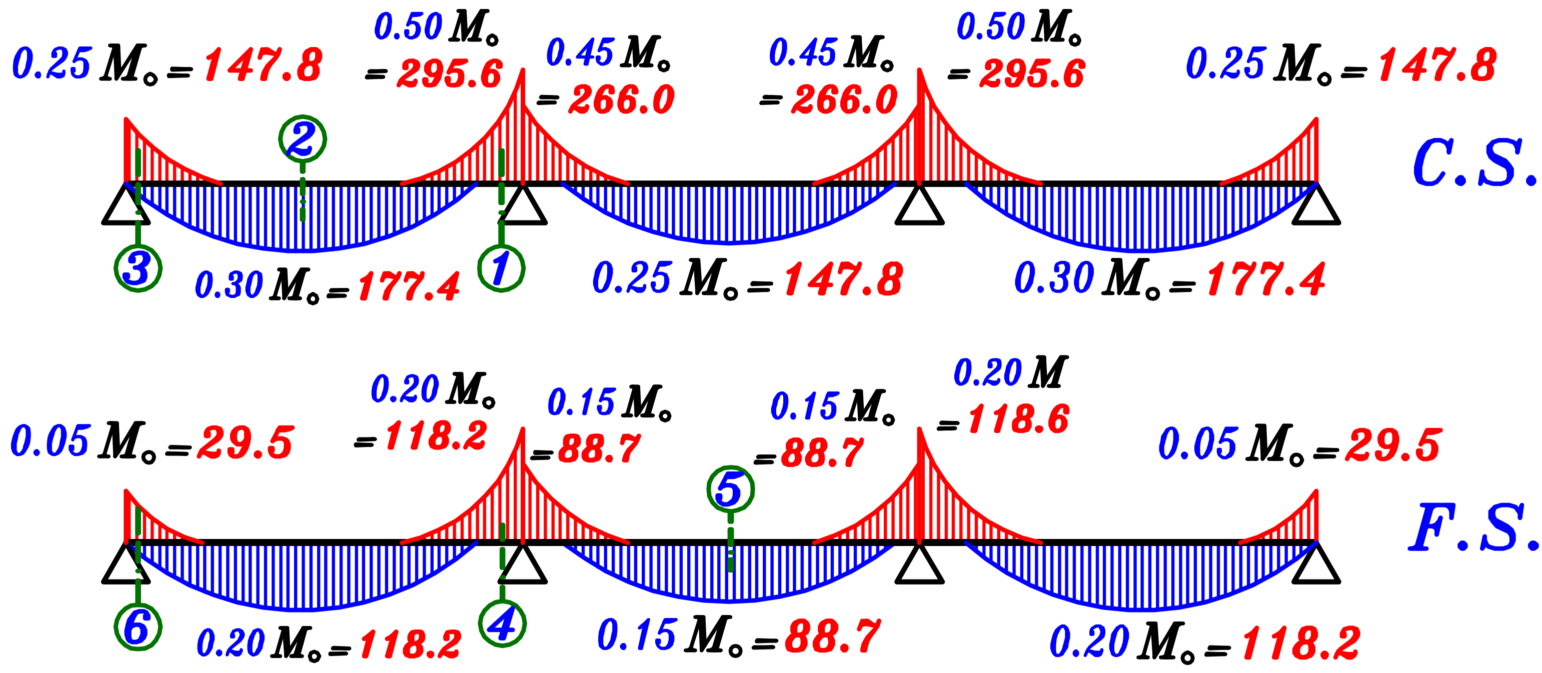
5- Distribute the B.M. (M_o) on C.S. & F.S.



Column Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0$ m

Field Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0$ m

$\boxed{M_o = 591.3 \text{ kN.m}}$ Long Direction

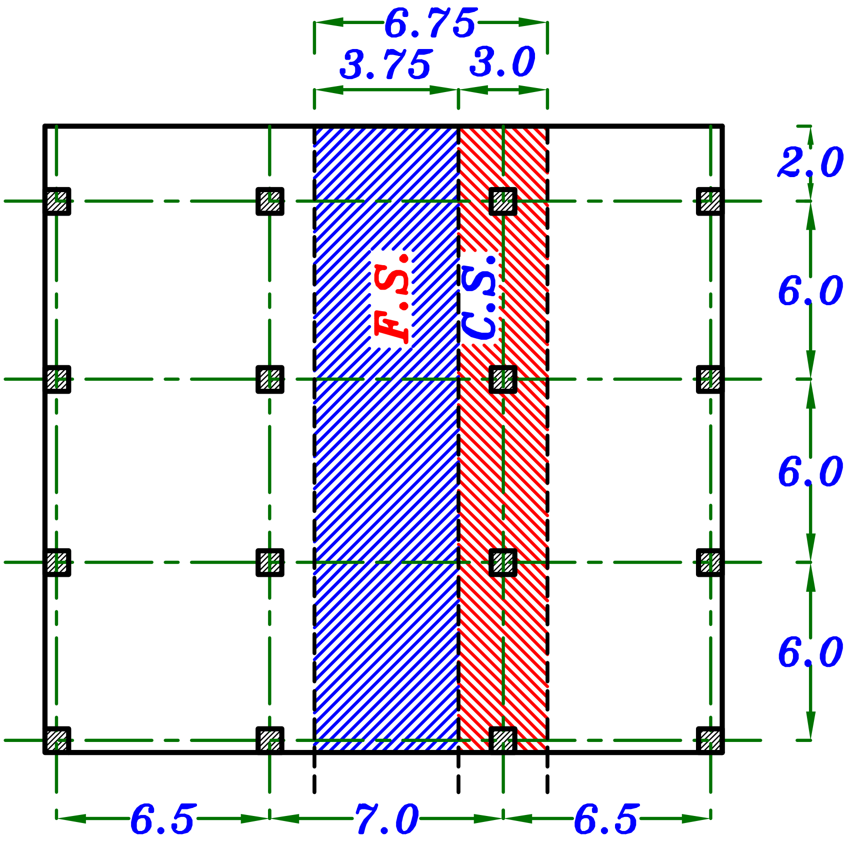


6– Design of sections.

$d = t_s - 30\ mm = 220 - 30 = 190\ mm$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm²/b)	As (mm²/m)	No. of bars/m
Column Strip	1	295.6	3000	190	3.32	0.769	5058	1686	7Φ18\m
	2	177.4	3000	190	4.28	0.812	2847	958	5Φ16\m
	3	147.8	3000	190	4.69	0.822	2365	788	7Φ12\m
Field Strip	4	118.2	3000	190	5.24	0.826	1883	628	6Φ12\m
	5	88.7	3000	190	6.05	0.826	1413	471	5Φ12\m
	6	29.5	3000	190	10.48	0.826	470	157	5Φ12\m

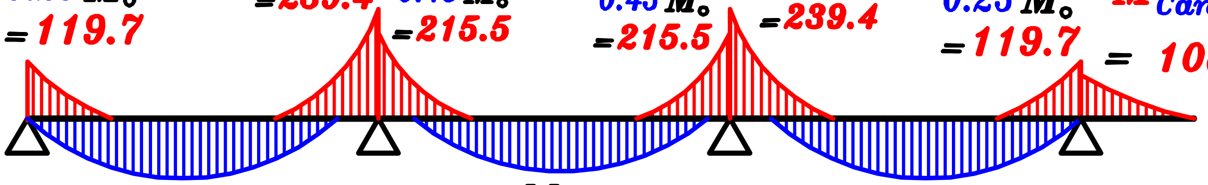
Short Direction.



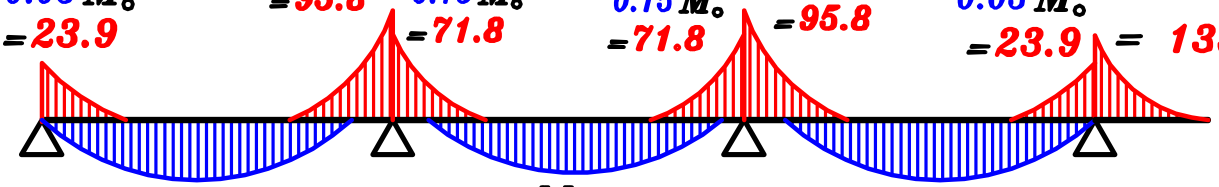
$$\text{Column Strip width} = \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

$$\text{Field Strip width} = L_1 - \frac{L_2}{2} = 6.75 - \frac{6.0}{2} = 3.75\ m$$

$\boxed{M_o = 478.9\ kN.m}$ Short Direction



C.S.
$(M_{C.S.})$

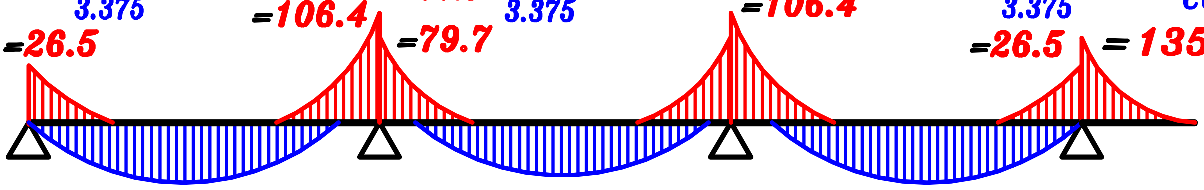


F.S.
$(M_{F.S.})$

$$\text{Modification Factor} = \frac{L_1 - L_2 \backslash 2}{L_1 \backslash 2} = \frac{3.75}{3.375}$$

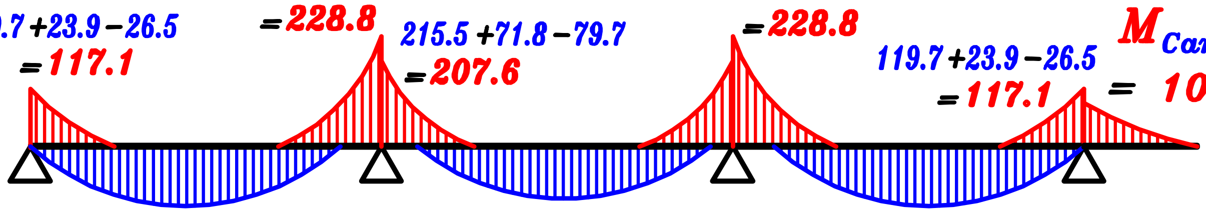
Modified Moment.

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{3.75}{3.375}}$$

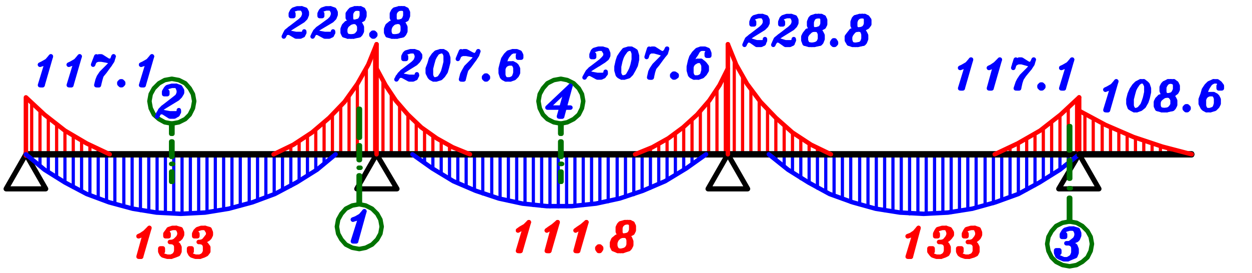


F.S.
$(M_{F.S.})_{mod.}$

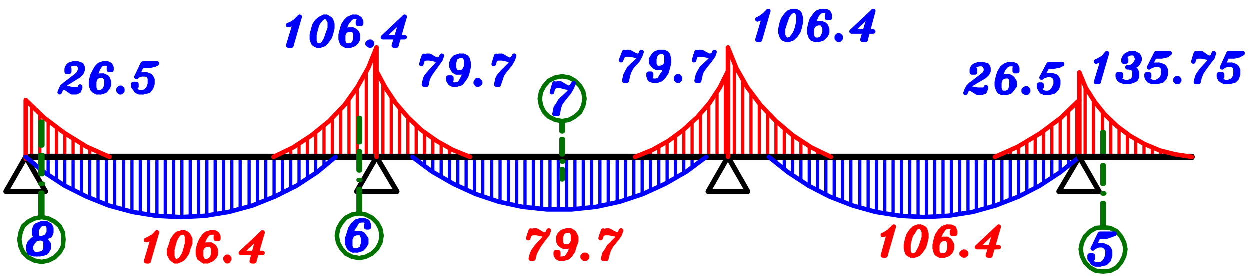
$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$



C.S.
$(M_{C.S.})_{mod.}$



C.S.
$(M_{c.s.})$ mod.



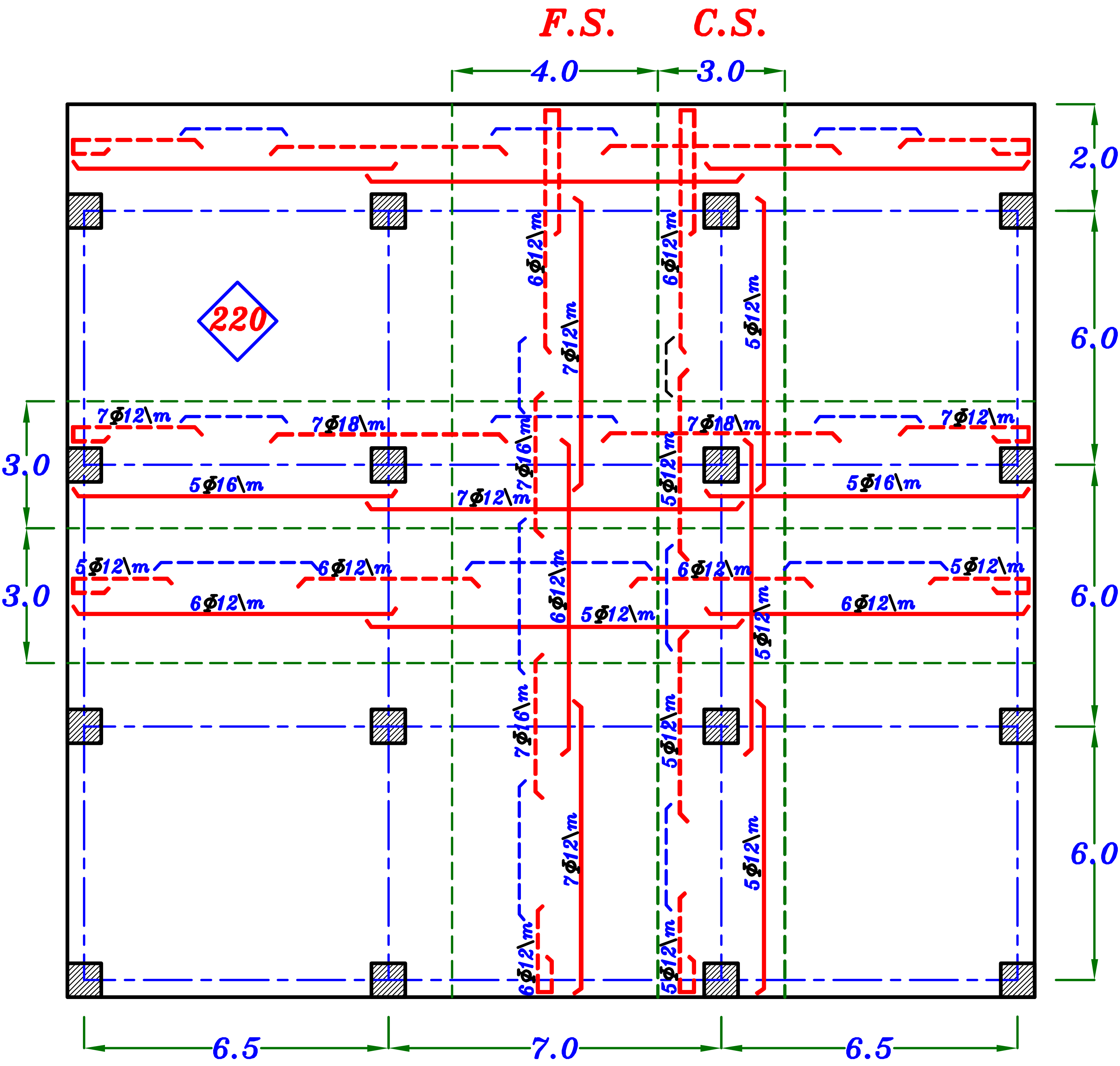
F.S.
$(M_{F.S.})$ mod.

6–Design of sections.

$d = t_s - 40 \text{ mm} = 220 - 40 = 180 \text{ mm}$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm²/b)	As (mm²/m)	No. of bars/m
Column Strip	1	228.8	3000	180	3.57	0.784	4053	1351	7Φ16\m
	2	133	3000	180	4.68	0.822	2248	749	7Φ12\m
	3	117.1	3000	180	4.99	0.826	1969	656.3	6Φ12\m
	4	111.8	3000	180	5.11	0.826	1880	627	6Φ12\m
Field Strip	5	135.75	3750	180	5.18	0.826	2282	608	6Φ12\m
	6	106.4	3750	180	5.85	0.826	1789	477	5Φ12\m
	7	79.7	3750	180	6.76	0.826	1340	357.4	5Φ12\m
	8	26.5	3750	180	11.72	0.826	445.6	119	5Φ12\m

7- Details of RFT.



Example.

The given plan shows general layout of a Flat slab Floor

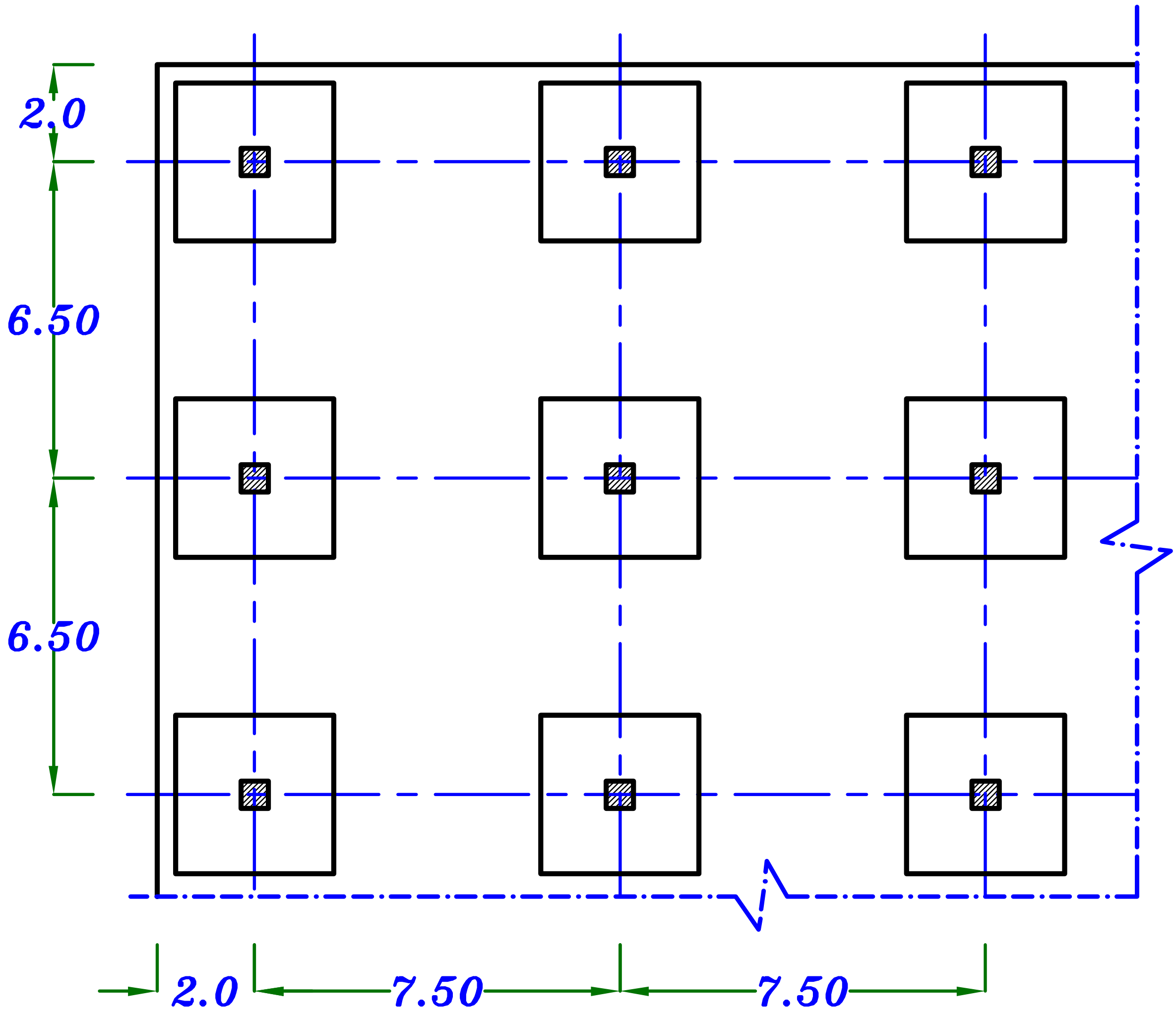
The column height 4.50 m

Data. $F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

F.C.+ Walls $= 3.50\ kN\backslash m^2$, L.L. $= 3.0\ kN\backslash m^2$

Req.

1. Using empirical method calculate the moments For both the Field strip and the column strip in both directions.
2. design the sections of the slab. and draw details of reinforcement in plan.



part plan

Solution.

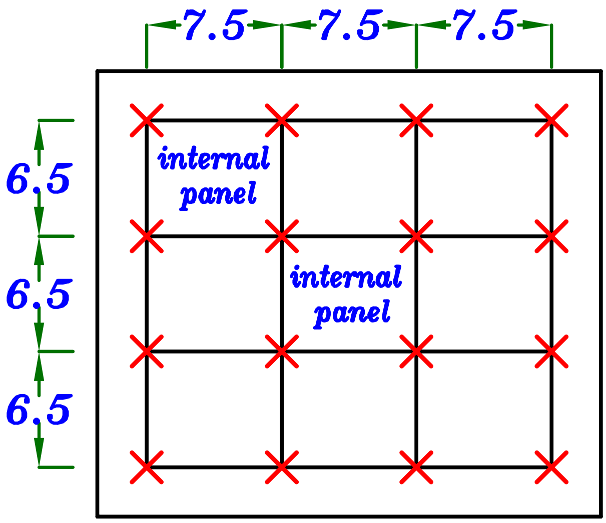
1-Concrete Dimensions.

Column dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{4500}{15} = 300\ mm \\ \dfrac{L_1}{20} = \dfrac{7500}{20} = 375\ mm \end{cases}$$

$$\boxed{b_{Col.} = 400\ mm}$$

$(400 * 400)$



Slab Thikness.

External panel لا يوجد

$L_1 = 7.5\ m$

For Internal panel

$$t_s = \frac{L_1}{40} = \frac{7500}{40} = 187.5\ mm = 200\ mm$$

$$\boxed{t_s = 200\ mm}$$

Drop Panel.

Take $t_d = \dfrac{t_s}{2} = 100\ mm$ رقم زوجي

Take Width of drop panel $X = \dfrac{L_2}{2} = 3.25\ m$ فى الإتجاهين

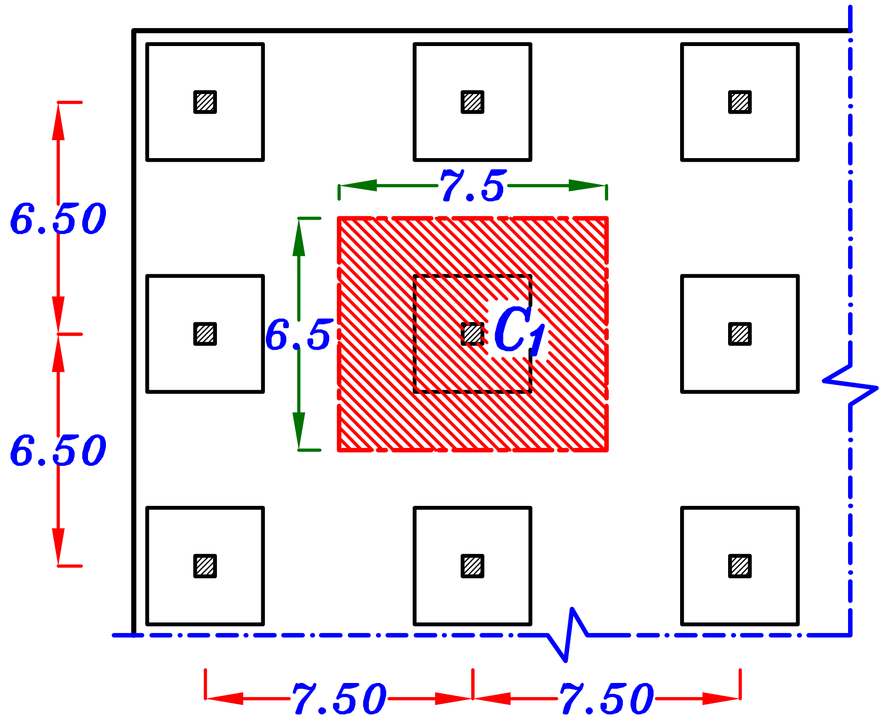
2-Loads on the Slab.

$$w_{s\,U.L.} = 1.4\left[\left(t_s + \frac{t_d}{4}\right)\gamma_c + F.C. + Wall\right] + 1.6\,(L.L.)$$

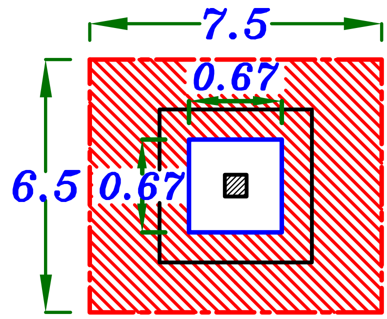
$$w_{s\,U.L.} = 1.4\left[\left(0.20 + \frac{0.1}{4}\right) * 25 + 3.50\right] + 1.6\,(3.0) = 17.57\ kN\backslash m^2$$

3 – Check Punching on interior column

كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى



C_1 Interior Column.



$$d = t_s + t_d - 30\ mm = 200 + 100 - 30 = 270\ mm = 0.27\ m$$

$$C + d = 0.40 + 0.27 = 0.67\ m$$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 17.57\ [7.5 * 6.5 - 0.67 * 0.67] = 848.65\ kN$$

$$A_p = (b_o * d) = (4 * 670) * 270 = 723600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{848.65 * 10^3}{723600} * 1.15 = 1.348\ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\delta_c}} = 0.316 \sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

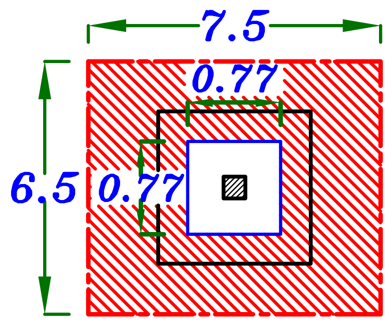
$$\boxed{q_{pu} > q_{pcu}} \longrightarrow$$

Unsafe punching

Increase dimensions of the column

C_1 Interior Column.

Take the Column (500 * 500)



$$d = t_s + t_d - 30\ mm = 200 + 100 - 30 = 270\ mm = 0.27\ m$$

$$C + d = 0.50 + 0.27 = 0.77\ m$$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1 + d)(C_2 + d) \right]$$

$$Q_{pu} = 17.57 \left[7.5 * 6.5 - 0.77 * 0.77 \right] = 846.12\ kN$$

$$A_p = (b_o * d) = (4 * 770) * 270 = 831600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{846.12 * 10^3}{831600} * 1.15 = 1.17\ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

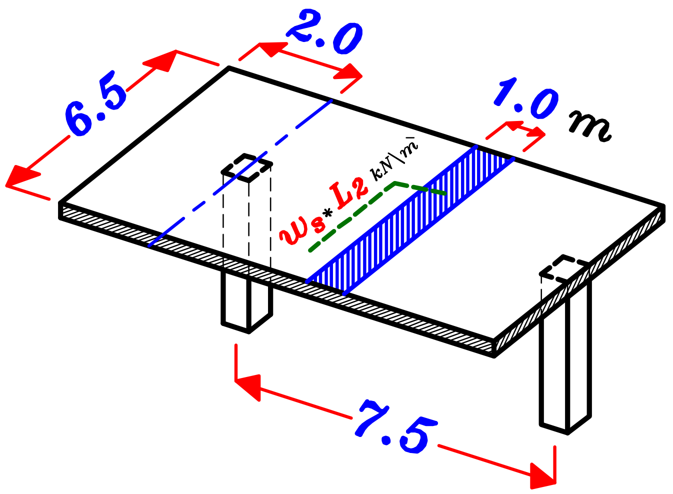
$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.

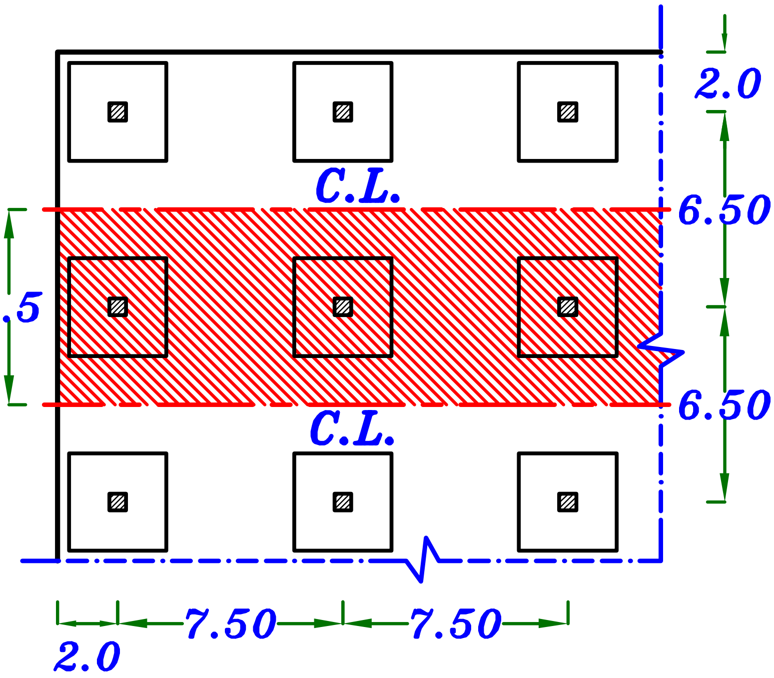
4– Take a Strips in the slabs at the long and short directions.
The strip width From C.L. the slab to C.L. the slab.

Strip at Long Direction.

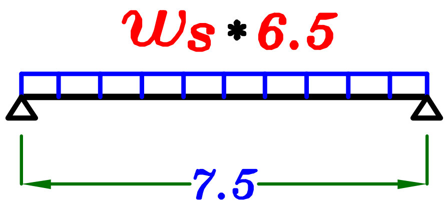
Span $= 7.5\ m$

Width $= 6.5\ m$





Moment in Long Direction.



$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8} = \frac{(17.57 * 6.5)\left(7.5 - \frac{2}{3} * 0.5\right)^2}{8}$$

$$\boxed{M_o = 733.21\ kN.m}$$ Long Direction

Cantilever Moment.

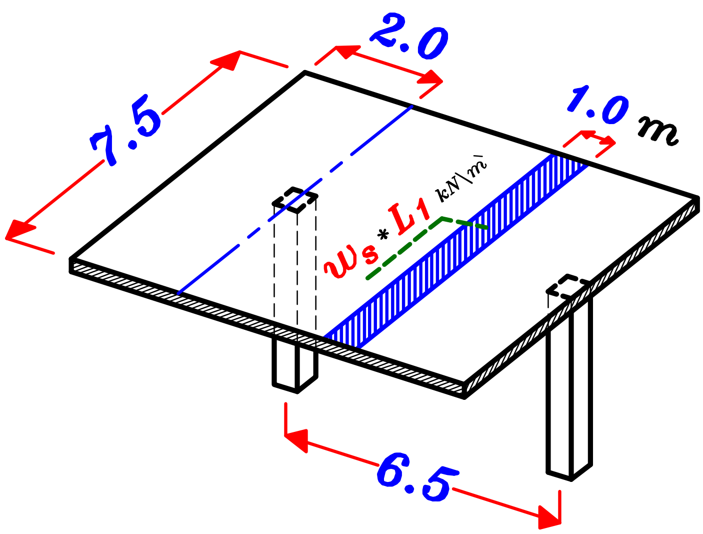
$$M_{Cant.}(C.S.) = \frac{w_s * (L_c)^2}{2} * b_{C.S.} = \frac{17.57 * (2.0)^2}{2} * 3.25 = 114.20\ kN.m$$

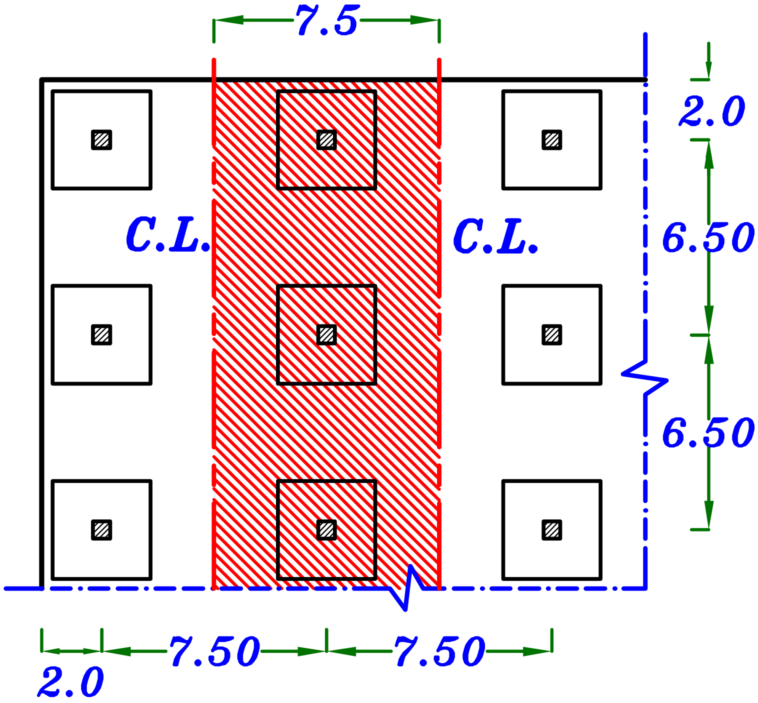
$$M_{Cant.}(F.S.) = \frac{w_s * (L_c)^2}{2} * b_{F.S.} = \frac{17.57 * (2.0)^2}{2} * 3.25 = 114.20\ kN.m$$

Strip at Short Direction.

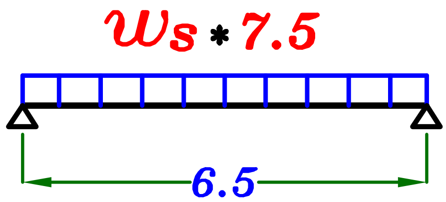
Span $= 6.5\ m$

Width $= 7.5\ m$





Moment in Short Direction.



$$M_o = \frac{(w_s * L_1)\left(L_2 - \frac{2}{3}D\right)^2}{8} = \frac{(17.57 * 7.5)\left(6.5 - \frac{2}{3} * 0.5\right)^2}{8}$$

$$\boxed{M_o = 626.39\ kN.m}$$ Short Direction

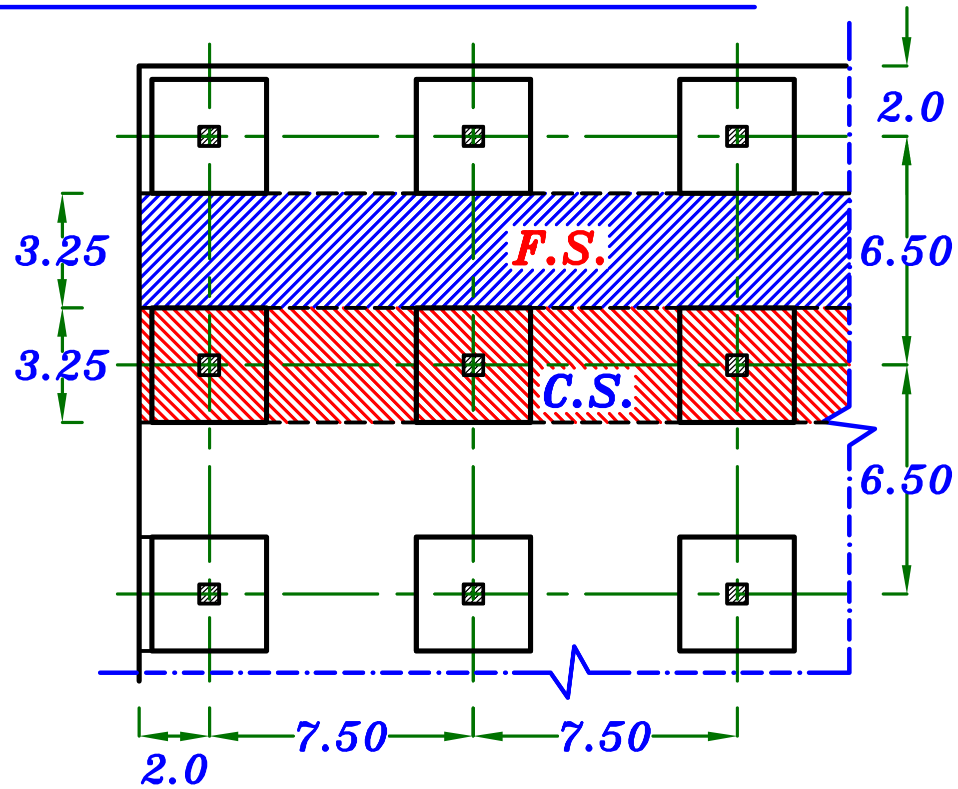
Cantilever Moment.

$$M_{Cant.}(C.S.) = \frac{w_s * (L_c)^2}{2} * b_{C.S.} = \frac{17.57 * (2.0)^2}{2} * 3.25 = 114.20\ kN.m$$

$$M_{Cant.}(F.S.) = \frac{w_s * (L_c)^2}{2} * b_{F.S.} = \frac{17.57 * (2.0)^2}{2} * 4.25 = 149.34\ kN.m$$

5- Distribute the B.M. (M_o) on C.S. & F.S.

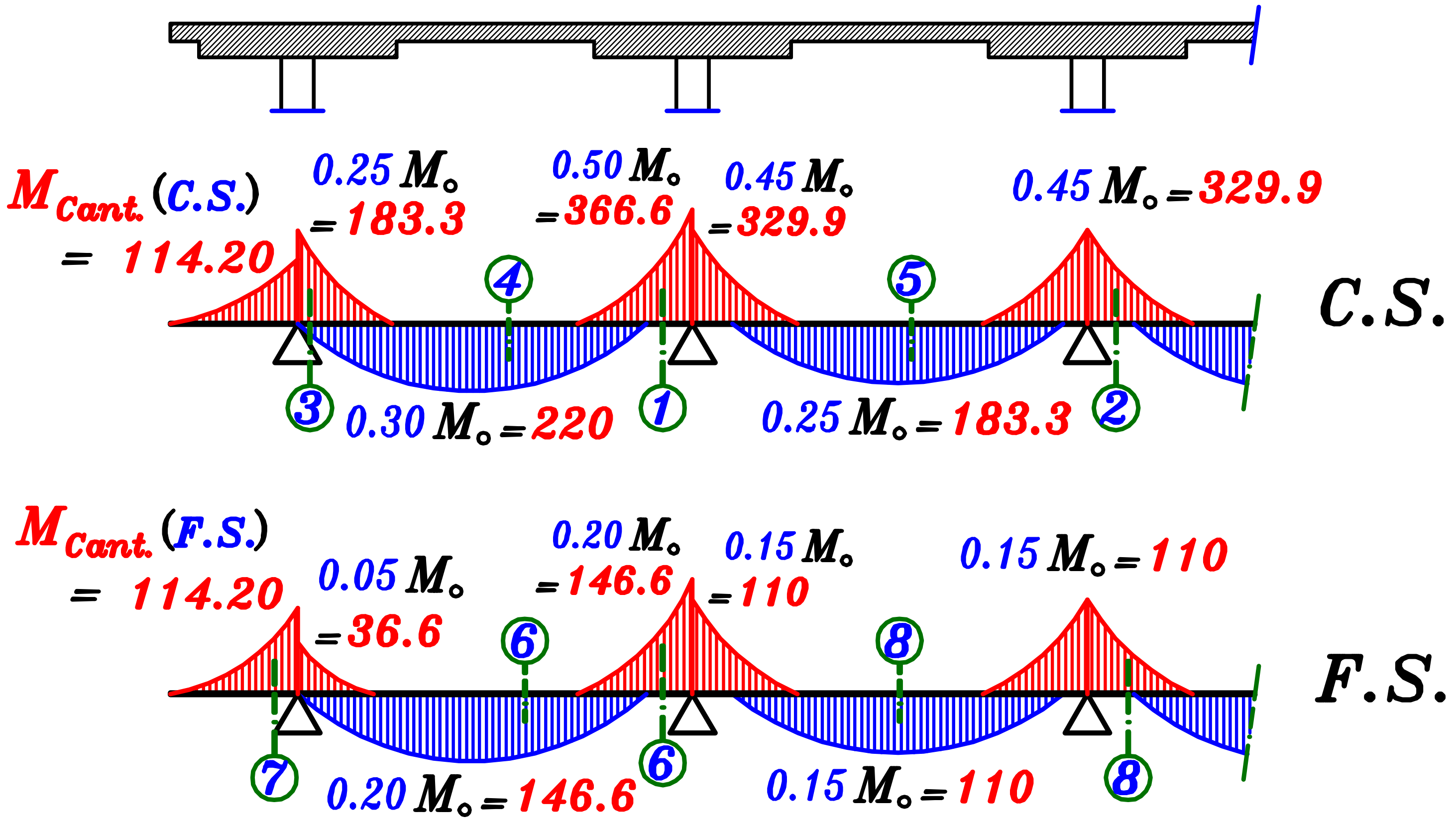
Long Direction.



Column Strip width $= X = \frac{L_2}{2} = \frac{6.5}{2} = 3.25$ m

Field Strip width $= L_2 - X = 3.25$ m

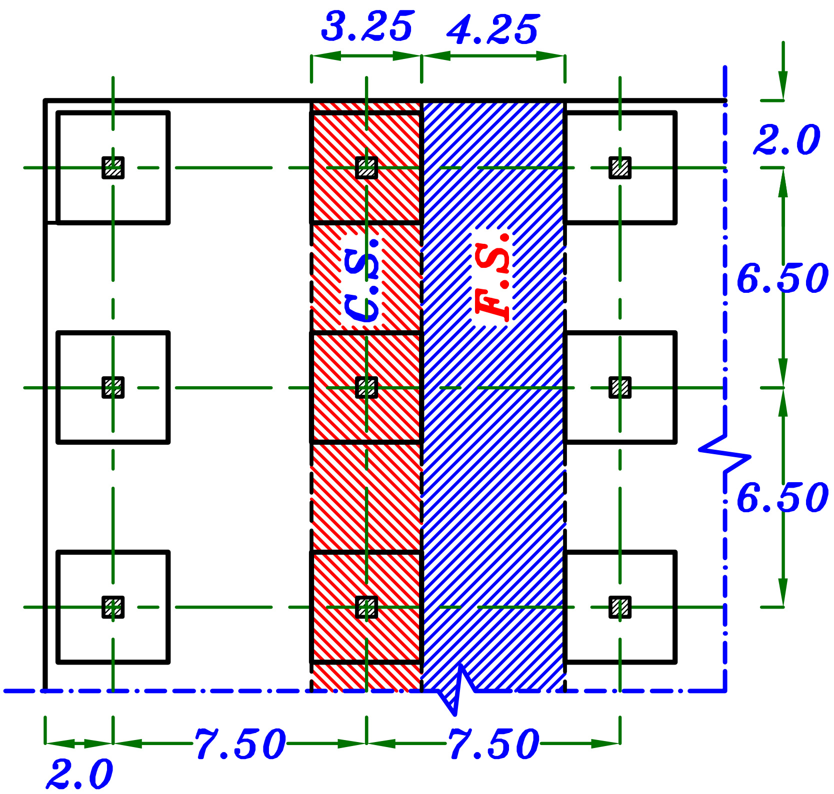
$\boxed{M_o = 733.21 \text{ kN.m}}$ Long Direction



6- Design of sections. $d = t_s - 30$ mm

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	366.6	3250	270	4.02	0.804	4691	1434	8ϕ16\m
	2	329.9	3250	270	4.23	0.811	4185	1287	7ϕ16\m
	3	183.3	3250	270	5.68	0.826	2283	702.4	7ϕ12\m
	4	220	3250	170	3.26	0.765	4699	1445	8ϕ16\m
	5	183.3	3250	170	3.58	0.785	3815	1174	6ϕ16\m
Field Strip	6	146.6	3250	170	4.00	0.803	2983	918	5ϕ16\m
	7	114.2	3250	170	4.53	0.819	2278	701	7ϕ12\m
	8	110	3250	170	4.62	0.820	2192	674.4	6ϕ12\m

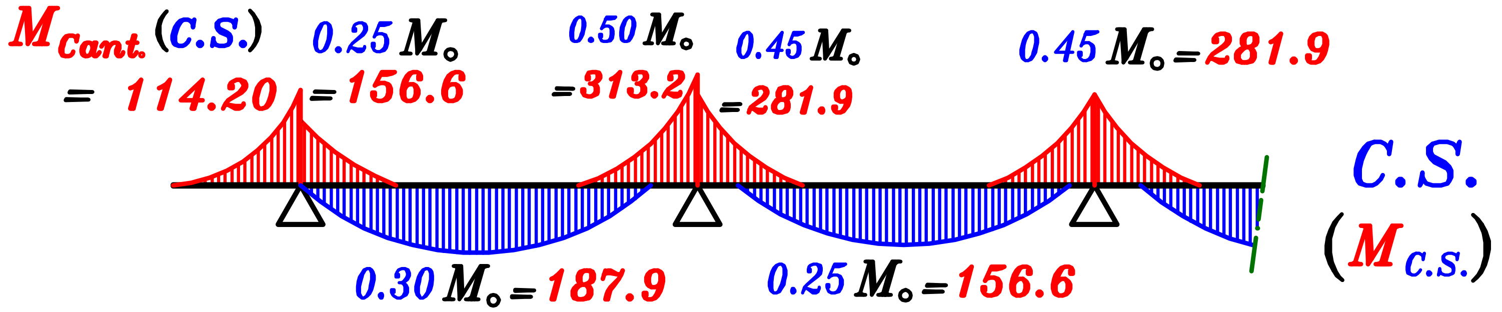
Short Direction.

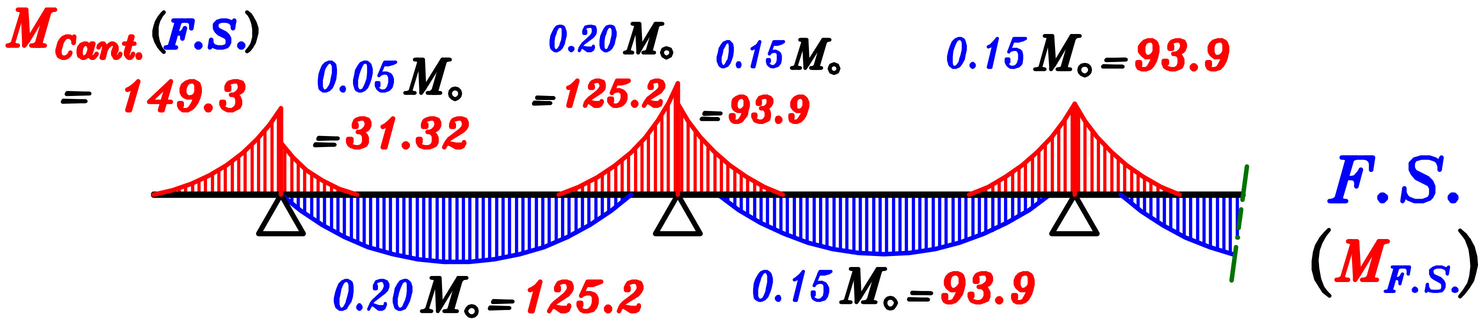


$$\text{Column Strip width} = X = \frac{L_2}{2} = \frac{6.5}{2} = 3.25 \text{ m}$$

$$\text{Field Strip width} = L_1 - X = 4.25 \text{ m}$$

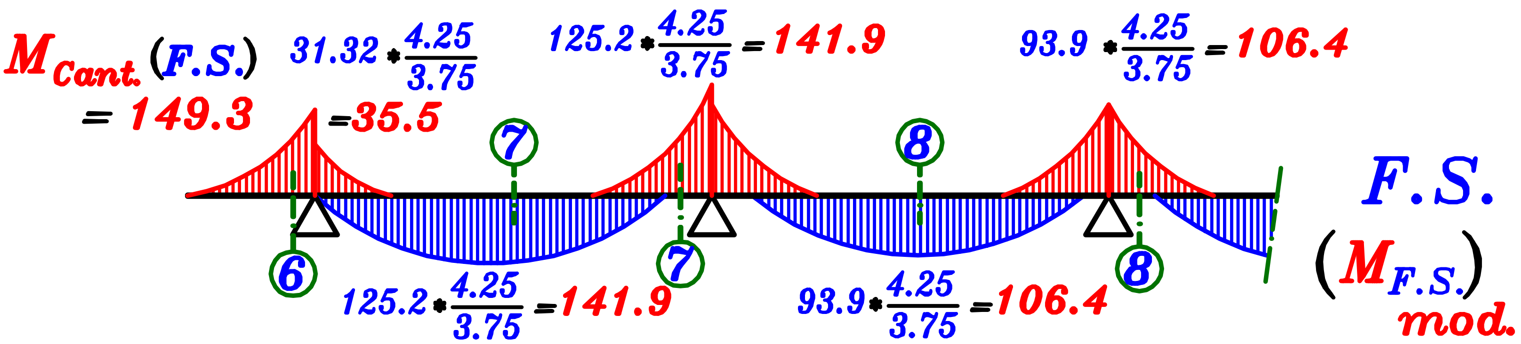
$$\boxed{M_o = 626.39 \text{ kN.m}}$$ Short Direction



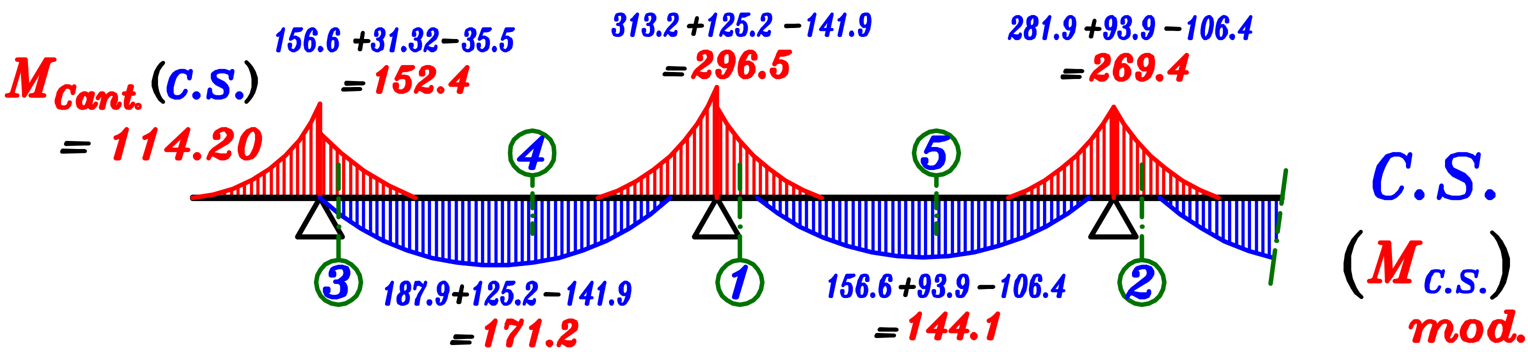


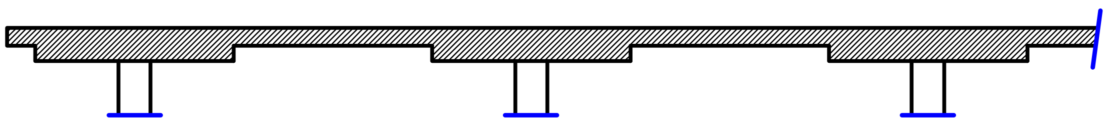
$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{4.25}{3.75}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.25}{3.75}}$$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

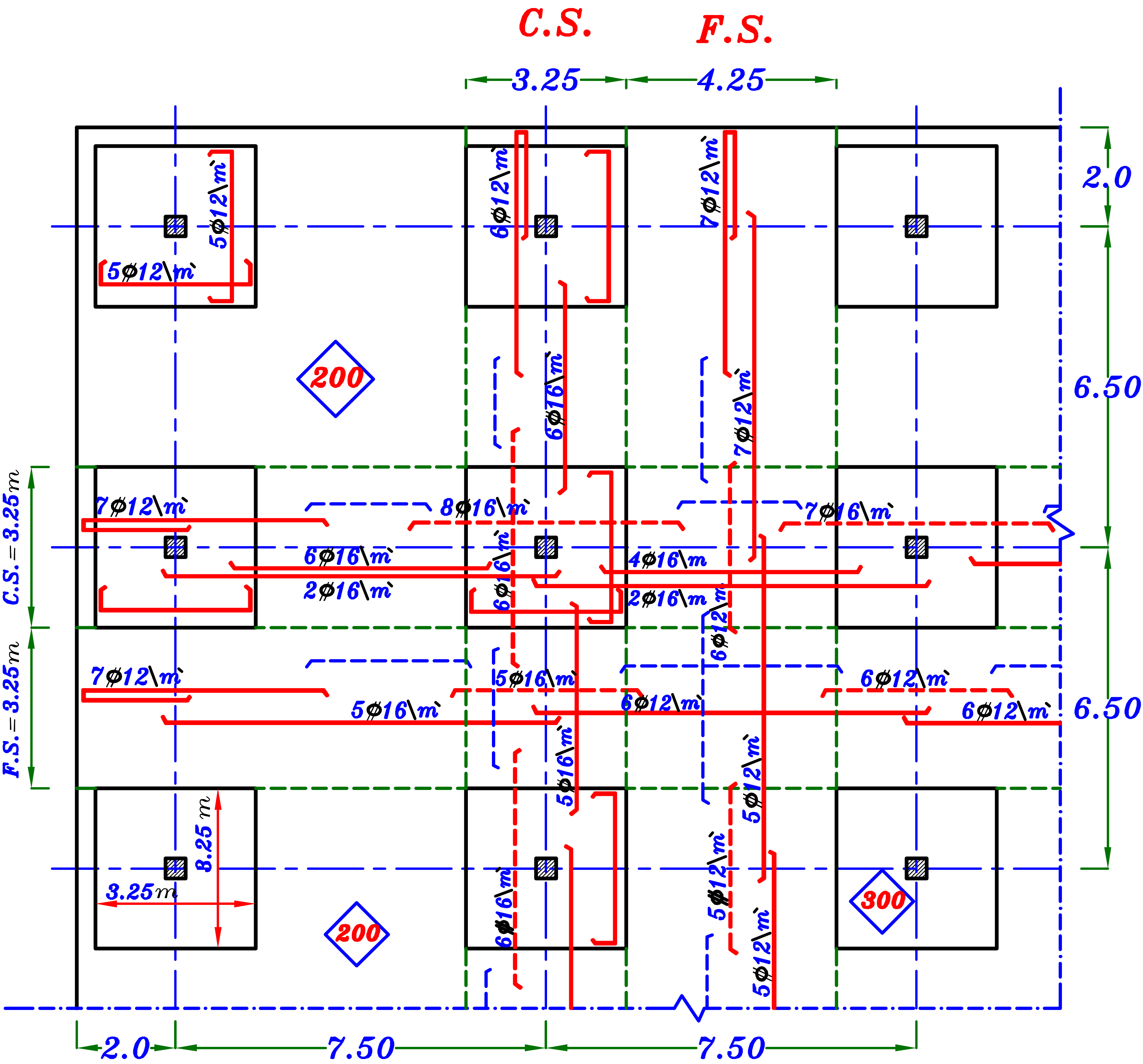




6- Design of sections. $d = t_s - 40$ mm

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm^2/b)	A_s (mm^2/m)	No. of bars/m
Column Strip	1	296.5	3250	260	4.30	0.813	3896	1198	6ϕ16\m
	2	269.4	3250	260	4.51	0.819	3514	1081	6ϕ16\m
	3	152.4	3250	260	6.00	0.826	1971	606	6ϕ12\m
	4	171.2	3250	160	3.48	0.780	3598	1107	6ϕ16\m
	5	144.1	3250	160	3.80	0.795	3146	968	5ϕ16\m
Field Strip	6	149.3	4250	160	4.26	0.812	3192	751	7ϕ12\m
	7	141.9	4250	160	4.38	0.816	3019	710	7ϕ12\m
	8	106.4	4250	160	5.06	0.826	2236	526	5ϕ12\m

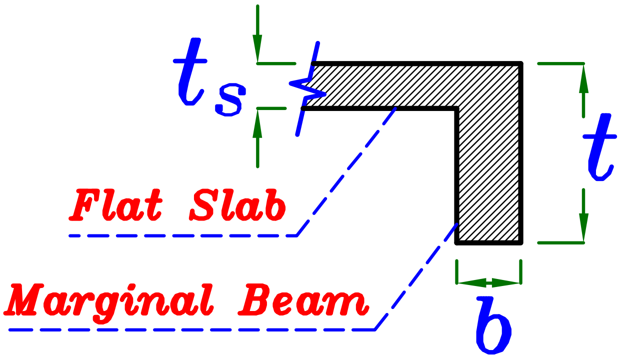
7- Details of RFT.

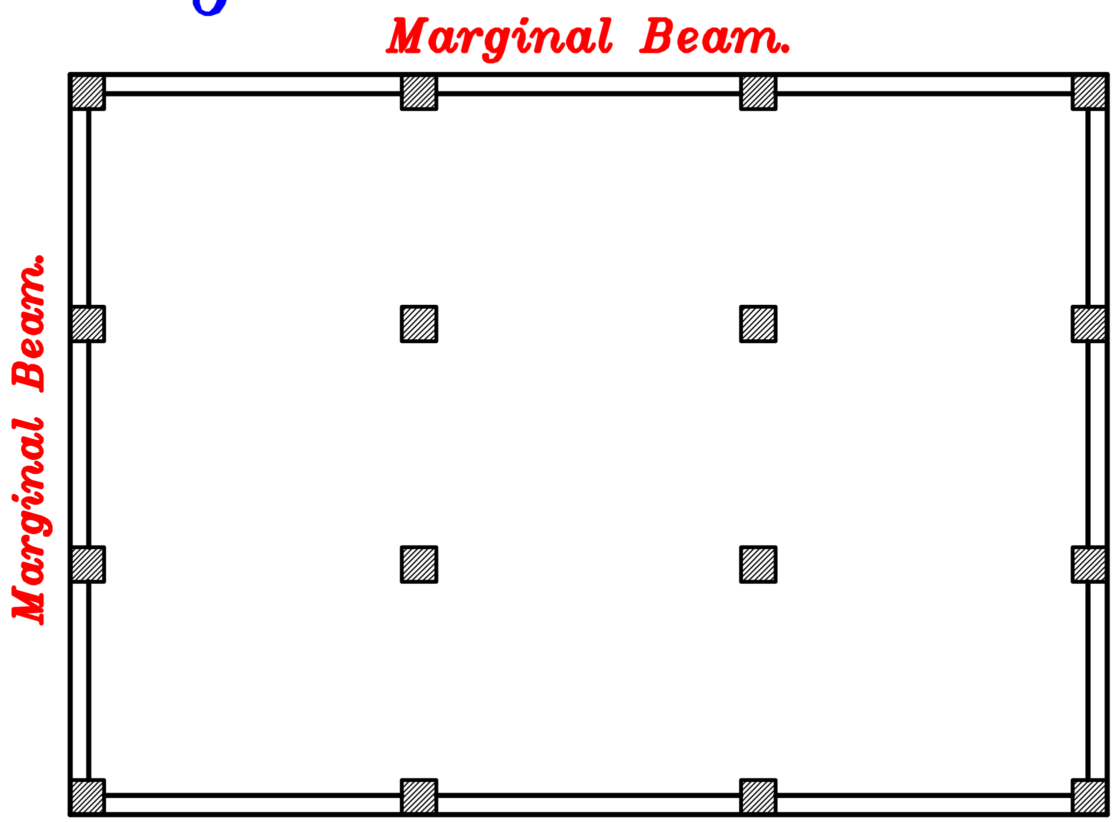


Design of Marginal Beam.

هى عباره عن كمره تكون على أطراف البلاطه الخارجيه .
و ممكن وضع هذه الكمره أو ترك البلاطه بدونها .
و لكى نضمن أن تعمل هذه الكمره على حمل البلاطة و ليست محموله عليها
يجب أن تكون الـ *Stiffness* للكمره أكبر بكثير من البلاطه .

لذا يجب أن تكون $\boxed{t \geqslant 3\,t_s}$





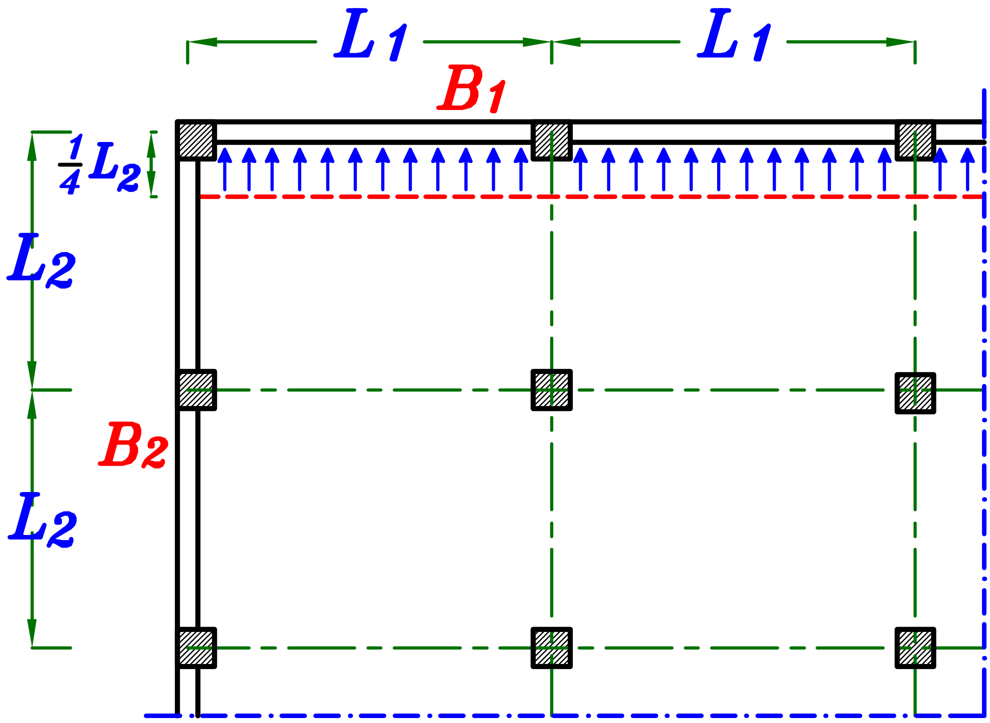
فوائد الـ *Marginal Beam* :

١- تحزيم المبنى لمقاومة الرياح و الزلازل .
٢- تقوية أطراف البلاطه .
٣- حمل حوائط الواجهه .

Loads on Marginal Beam.

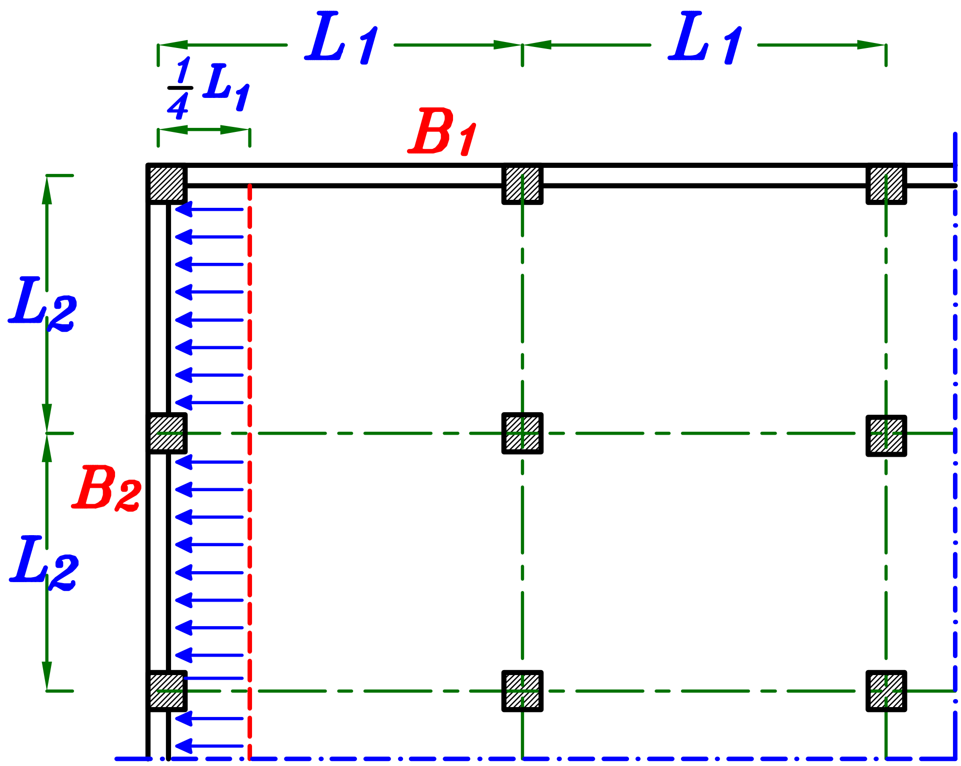
نعتبر أن الـ **Marginal Beam** تحمل $\frac{١}{٤}$ وزن الباكيه المجاوره لها .

B_1

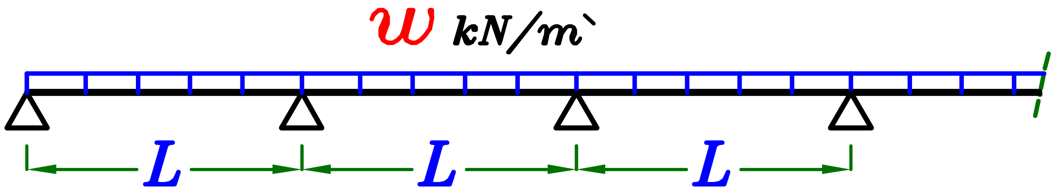


$$w_1 = O.W. + Wall + w_s * \left(\frac{1}{4} * L_2\right)$$

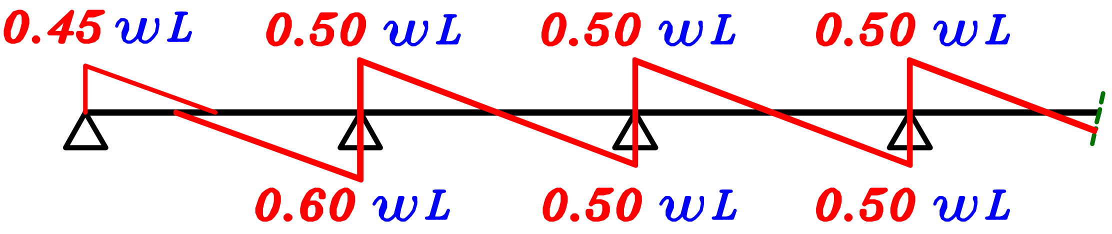
B_2



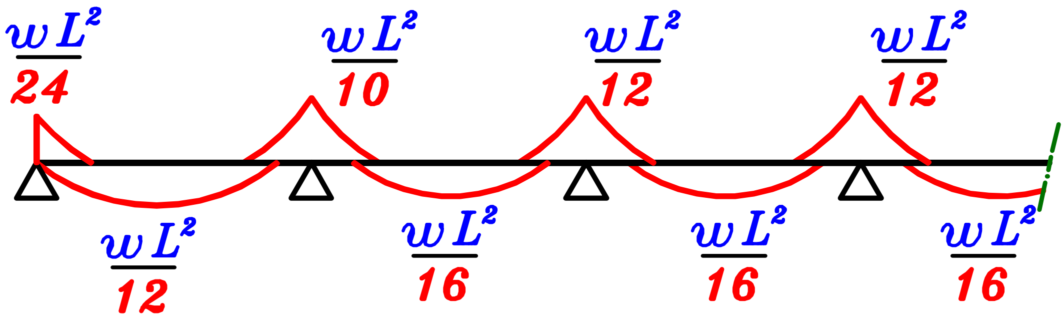
$$w_2 = O.W. + Wall + w_s * \left(\frac{1}{4} * L_1\right)$$



Loads on Marginal Beam

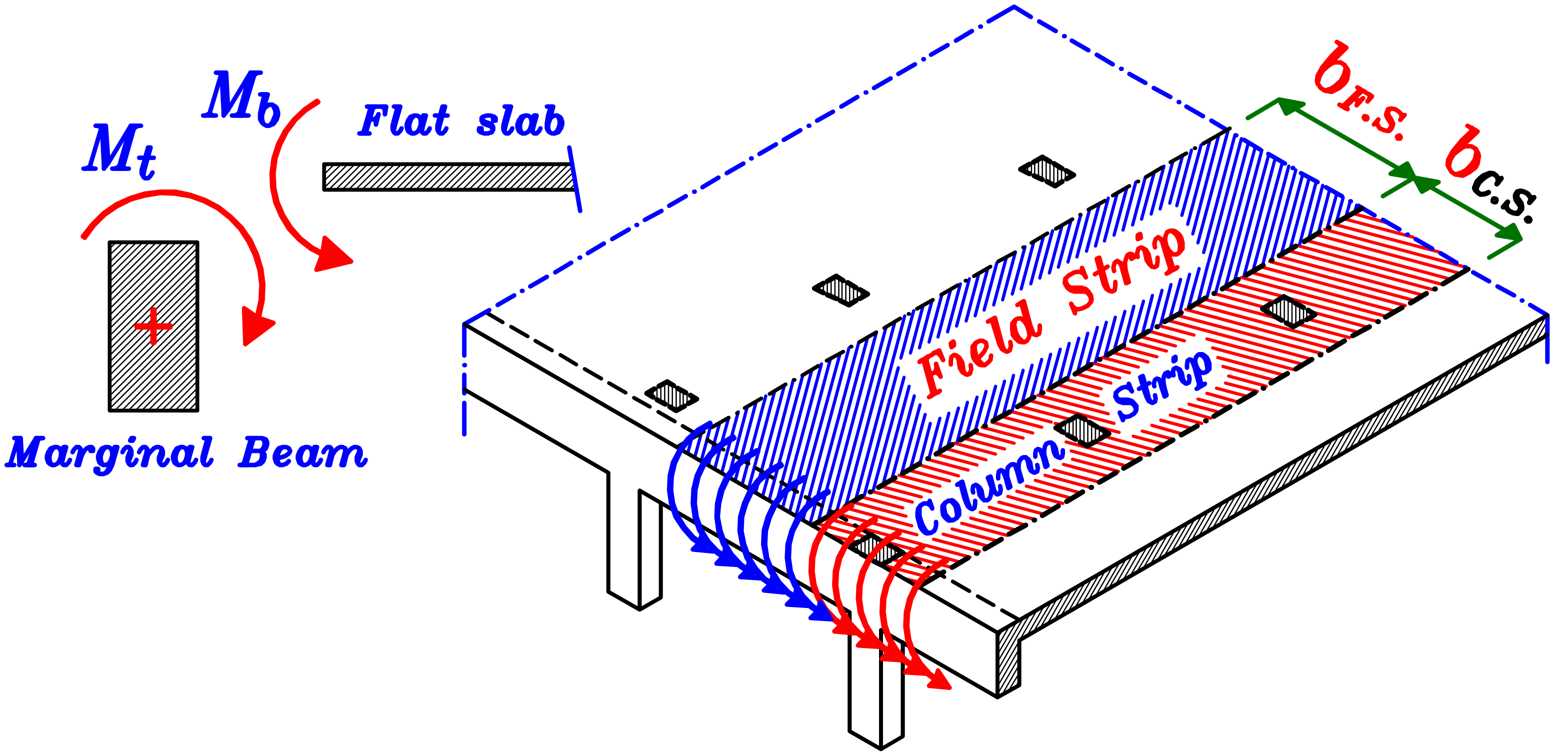


S.F.D. For Marginal Beam

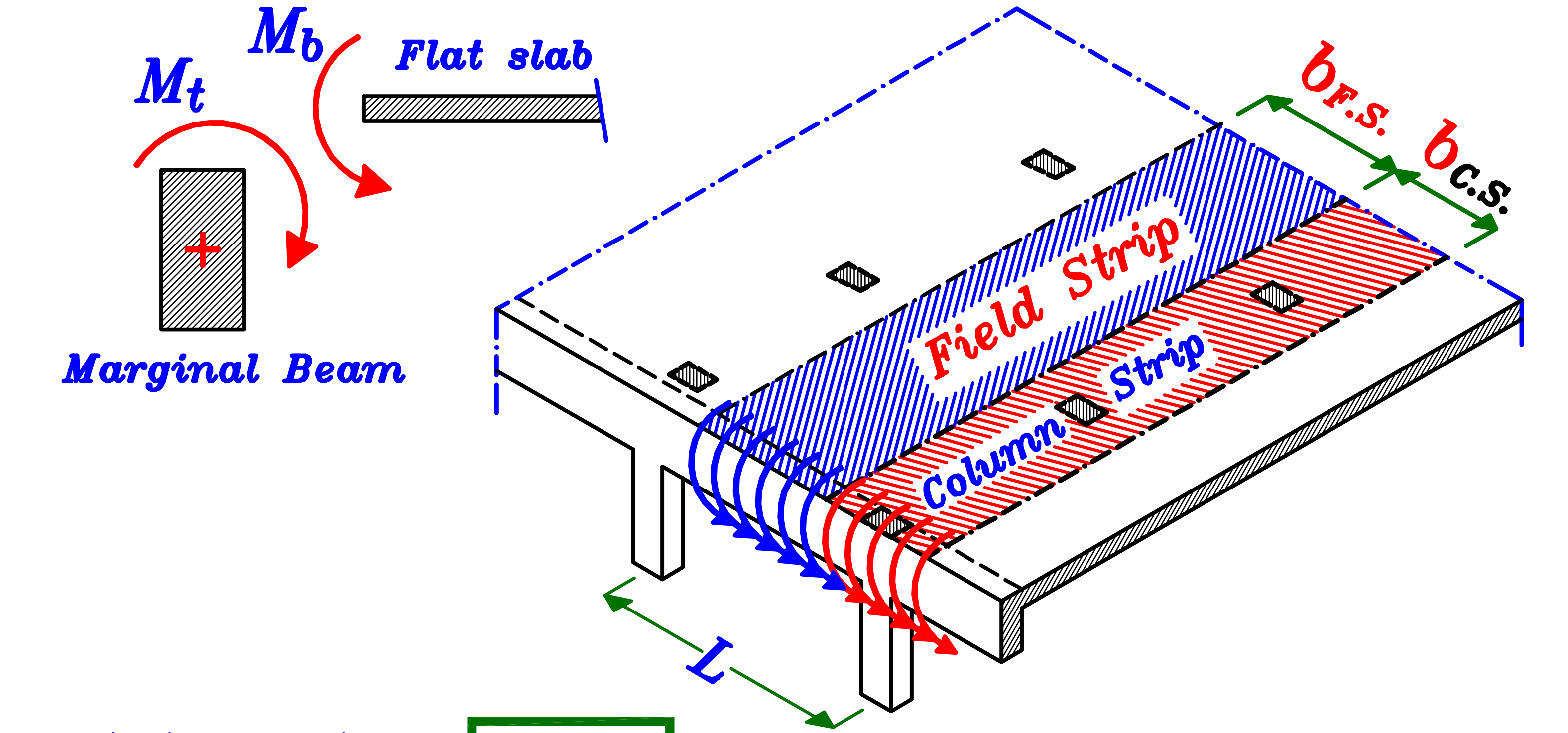


B.M.D. For Marginal Beam

Torsion on Marginal Beam.



ينتقل أخر **bending moment** موجود على الـ **F.S.** & **C.S.**
العمودين على الـ **Marginal Beam** الى **torsional moment**
على الـ **Marginal Beam**

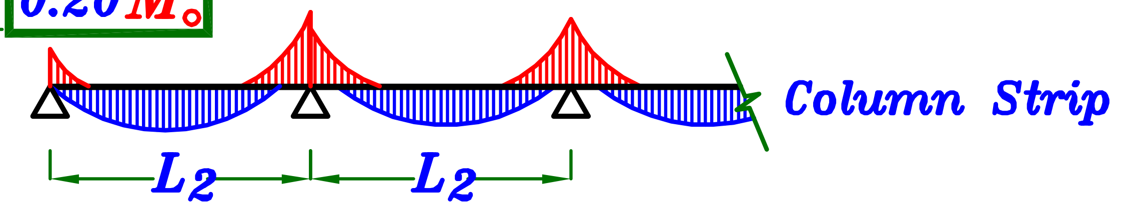


هذا العزم يتحول الى

Torsional moment

Marginal Beam على الـ

$\boxed{0.20\,M_o}$

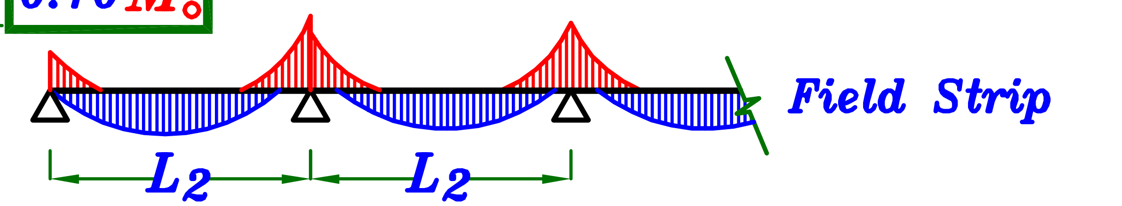


هذا العزم يتحول الى

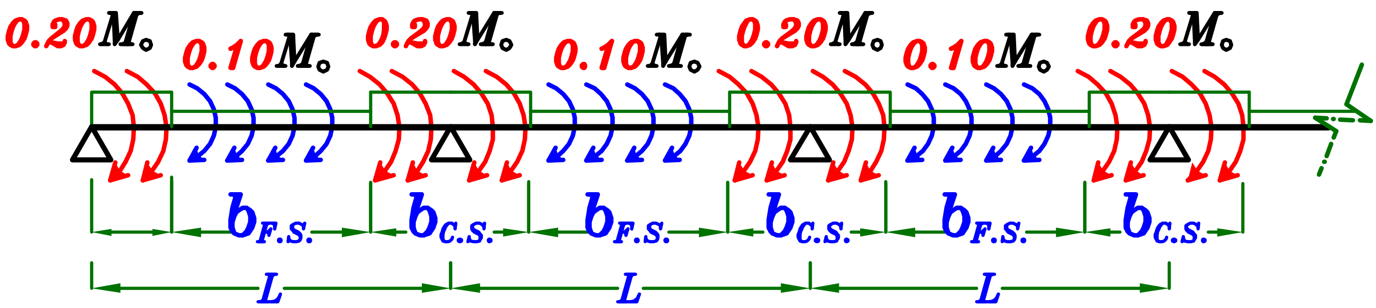
Torsional moment

Marginal Beam على الـ

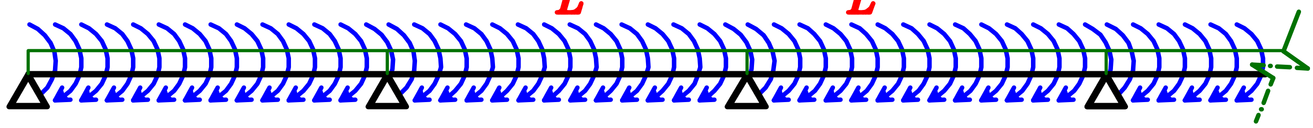
$\boxed{0.10\,M_o}$



Torsional moment on Marginal Beam.



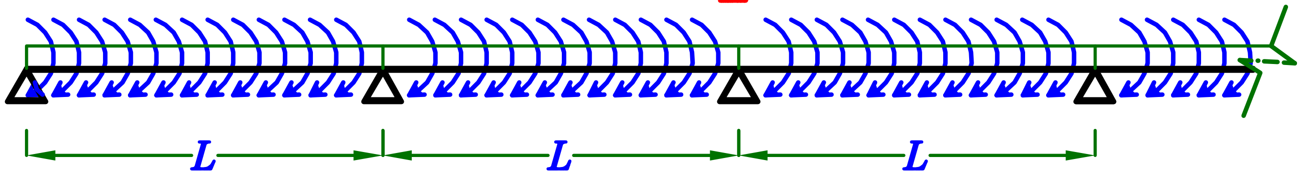
$$M_{t\,(av.)} \simeq \frac{(0.1+0.2)}{L}\,M_o \simeq \frac{0.3}{L}\,M_o \quad kN.m/m$$

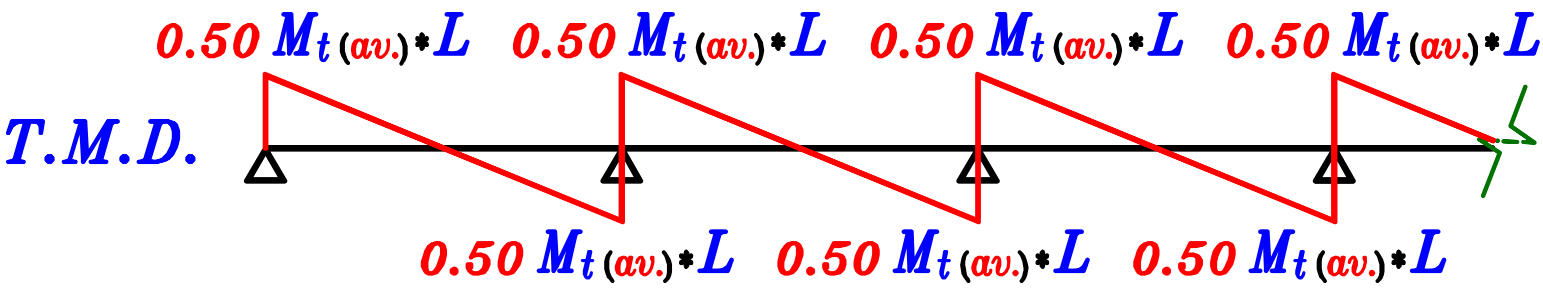


$$\boxed{M_{t\,(av.)} \simeq \frac{0.3}{L}\,M_o}$$

ممكن للتسهيل حساب متوسط للـ torsion moment

$$M_{t\,(av.)} \simeq \frac{0.3}{L} M_o$$

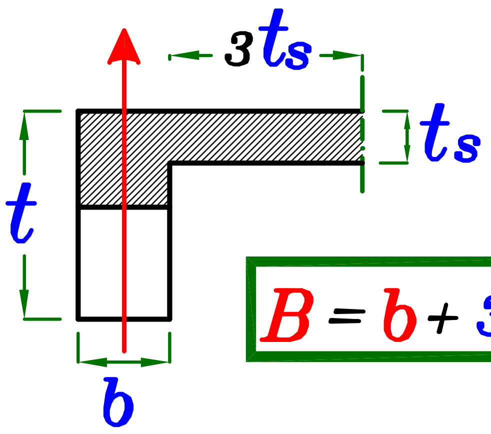




ملحوظه . في العمل ال torsional moment على الكمره أقل من $\frac{0.3}{L} M_o$
لان جزء من torsional moment يتحول مباشره الى bending moment على العمود

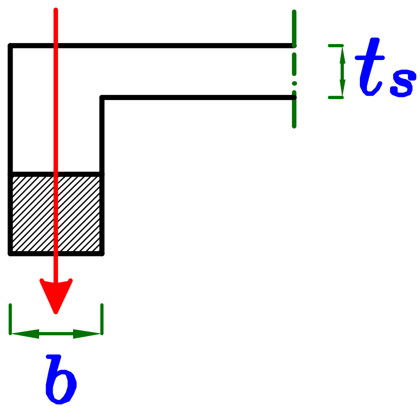
Design of Marginal Beam.

① Design due to Moment using C_1, J



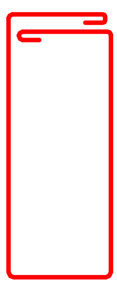
$$\boxed{B = b + 3t_s}$$

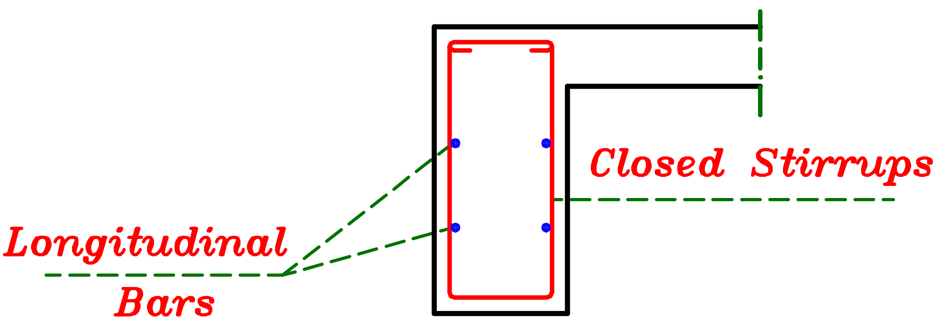
L-Sec.



R-Sec.

② Design due to Shear + Torsion





Example.

The given plan shows general layout of a Flat slab Floor

The column height 4.0 m

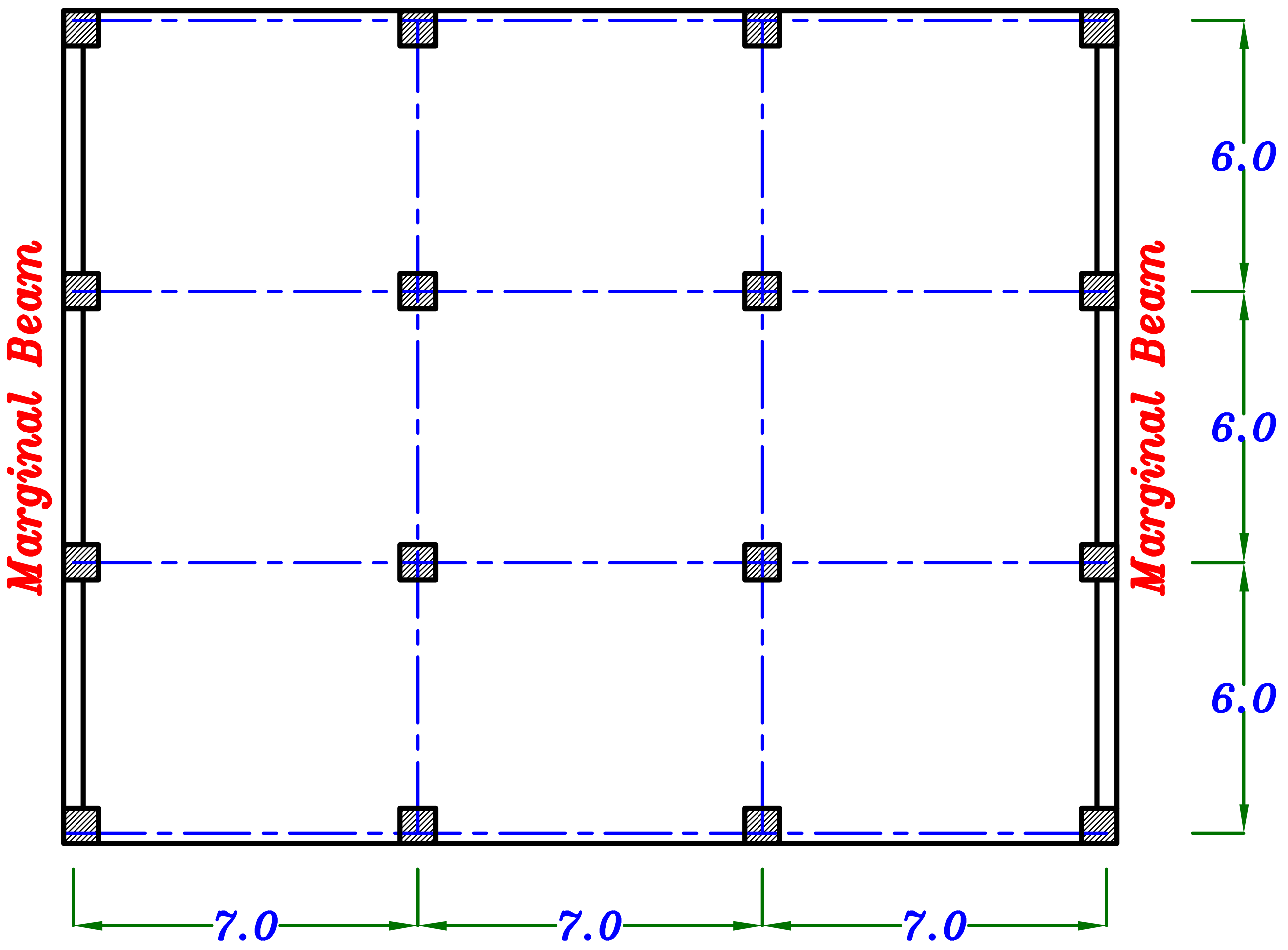
Data. $F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

F.C. $= 1.5\ kN\backslash m^2$, L.L. $= 4.0\ kN\backslash m^2$, Walls $= 1.5\ kN\backslash m^2$

Req.

① Using empirical method calculate the moments For both the Field strip and the column strip in the long direction.

② Design the Marginal Beam and draw details of reinforcement in elevation and cross sections.



Solution.

1- Concrete Dimensions.

Column dimensions.

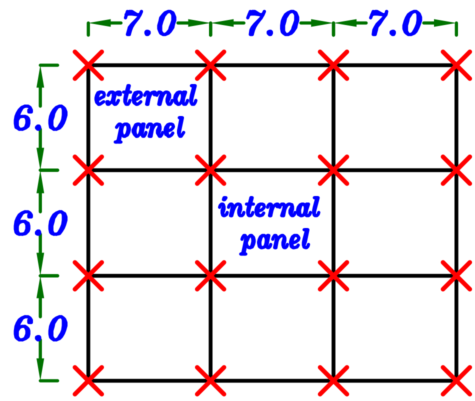
$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{4000}{15} = 266.6\ mm \\ \dfrac{L_1}{20} = \dfrac{7000}{20} = 350\ mm \end{cases}$$

$$\boxed{b_{Col.} = 350\ mm}$$

$(350 * 350)$

Slab Thikness.

$L_1 = 7.0\ m$



External panel $t_s = \dfrac{L_1}{32} = \dfrac{7000}{32} = 218.7\ mm$

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7000}{36} = 194.4\ mm$

$$\boxed{t_s = 220\ mm}$$

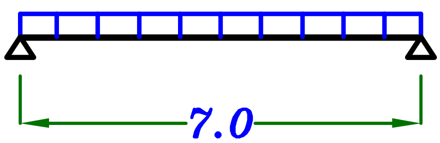
2- Loads on the Slab.

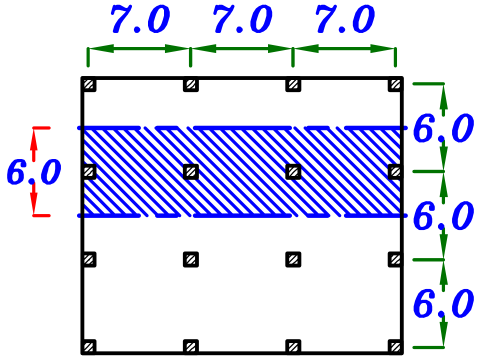
$$w_{s\,U.L.} = 1.4\left(t_s\, \gamma_c + F.C. + Walls\right) + 1.6\left(L.L.\right)$$

$$w_{s\,U.L.} = 1.4\left(0.22 * 25 + 1.50 + 1.50\right) + 1.6\left(4.0\right) = 18.30\ kN\backslash m^2$$

3- Take a Strips in the slabs at the long direction.

$w_s * 6.0$



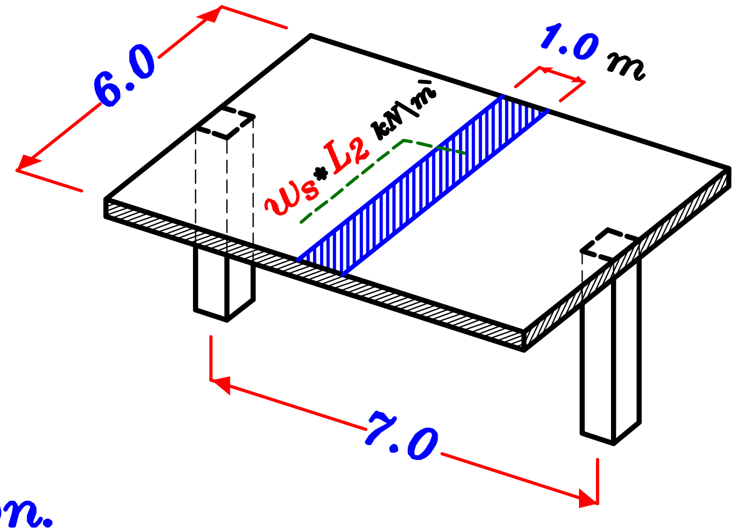


4- Calculate the moments on the strip.

Long Direction.

Span $= 7.0\ m$

Width $= 6.0\ m$



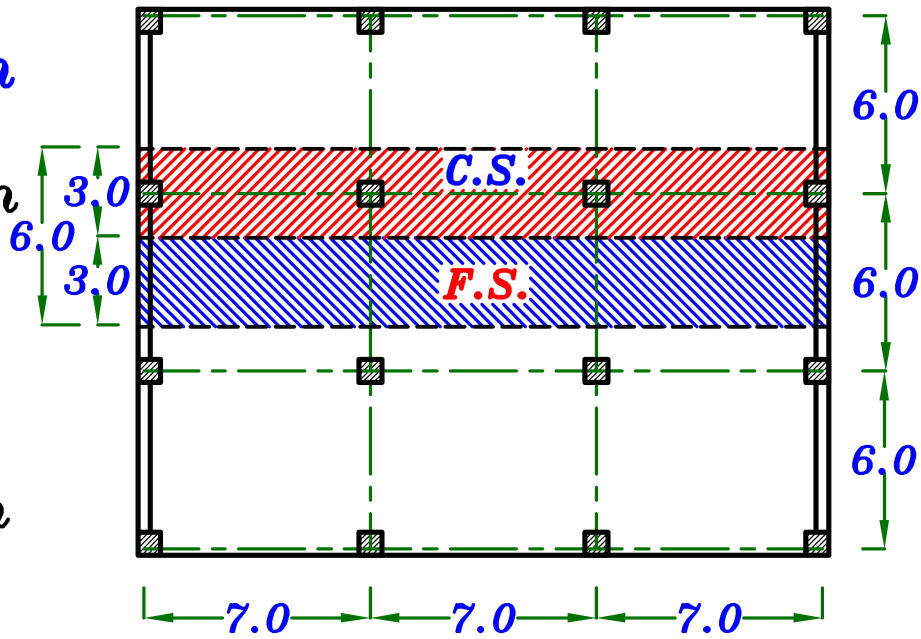
Moment in Long Direction.

$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8} = \frac{(18.30 * 6.0)\left(7.0 - \frac{2}{3} * 0.35\right)^2}{8}$$

$$\boxed{M_o = 628.4\ kN.m}$$ Long Direction

5- Distribute the B.M. (M_o) on C.S. & F.S.

Long Direction.



Column Strip width

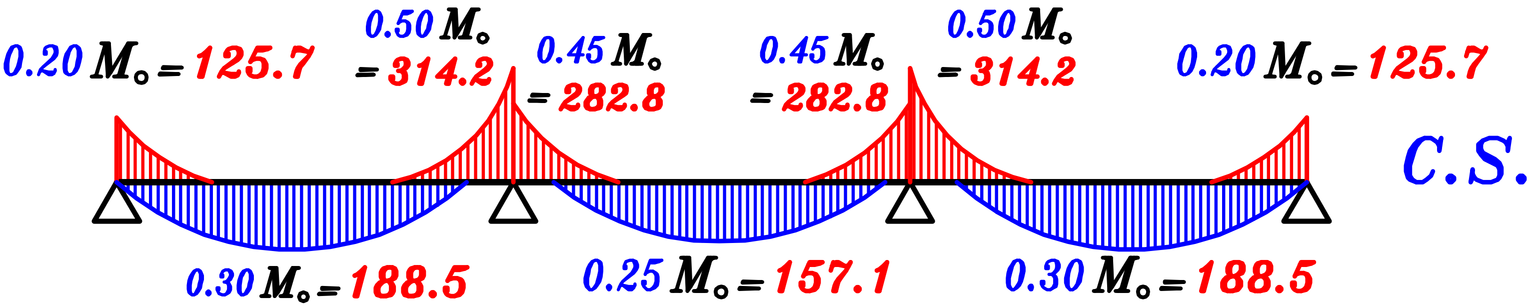
$$= \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

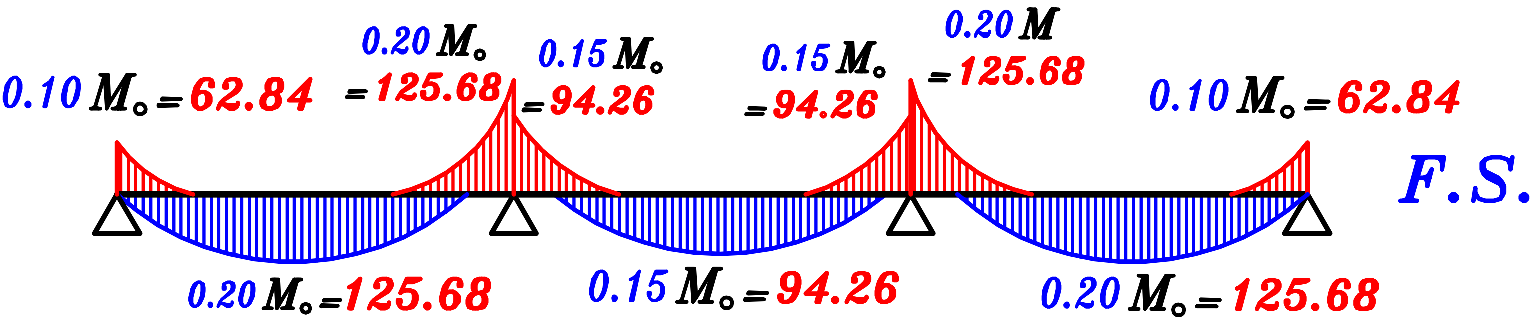
Field Strip width

$$= \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

$$\boxed{M_o = 628.4 \ kN.m}$$

Long Direction

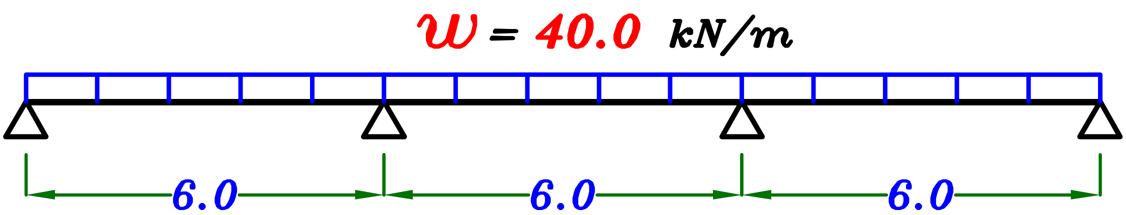


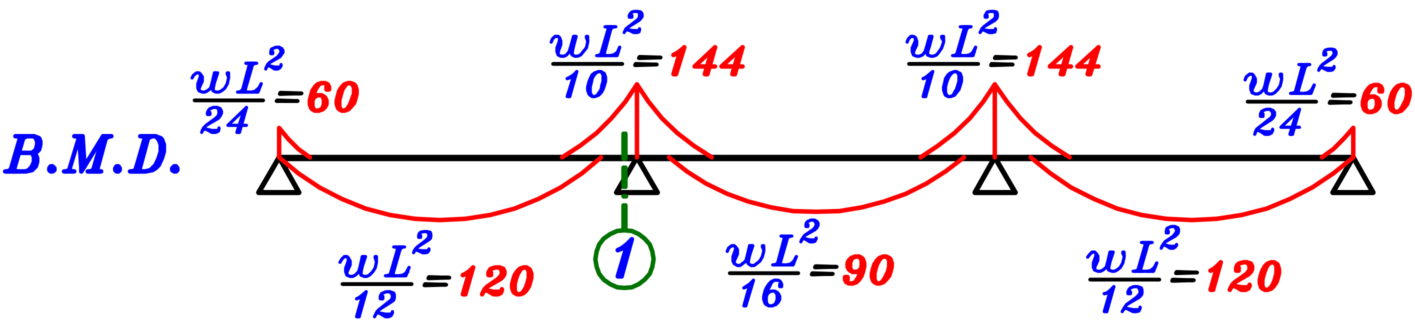


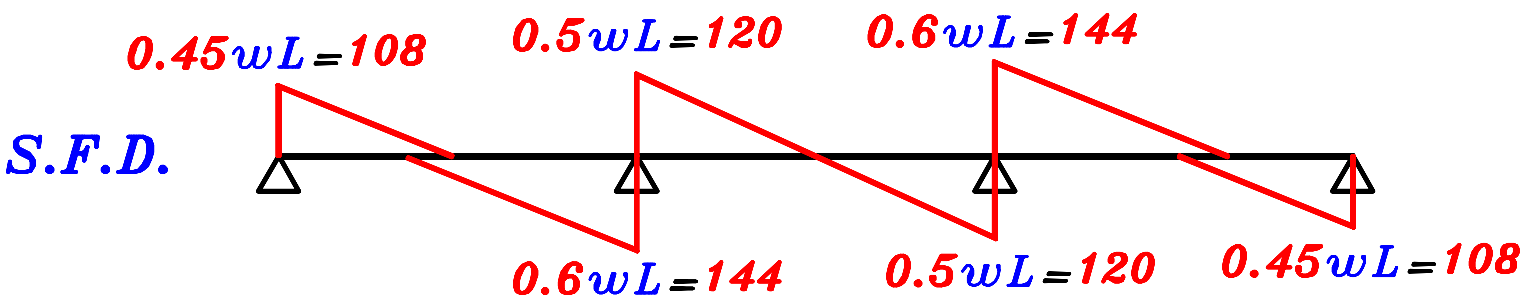
Loads on Marginal Beam.

Take o.w. of Marginal Beam = 8.0 kN/m (U.L.)

$$w = 0.W. + w_s * \left(\frac{1}{4} L_1\right) = 8.0 + 18.3 * \left(\frac{1}{4} * 7.0\right) = 40.0 \ kN/m$$

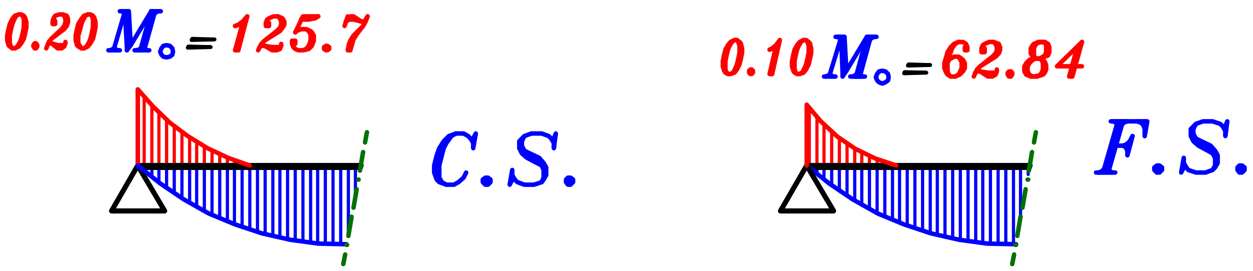




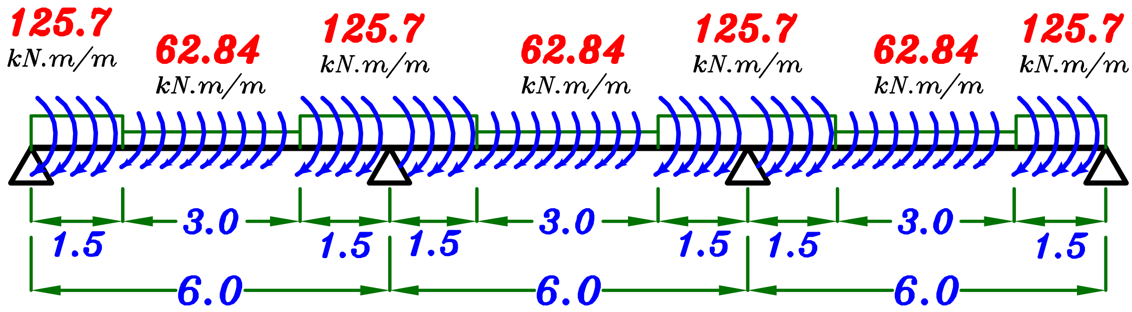


Torsion on Marginal Beam.

ينتقل أخر **bending moment** موجود على الـ **F.S. & C.S.**
العمودين على الـ **Marginal Beam** الى **torsional moment**
على الـ **Marginal Beam**



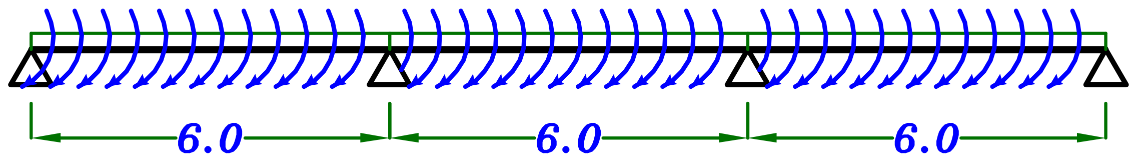
Torsional moment



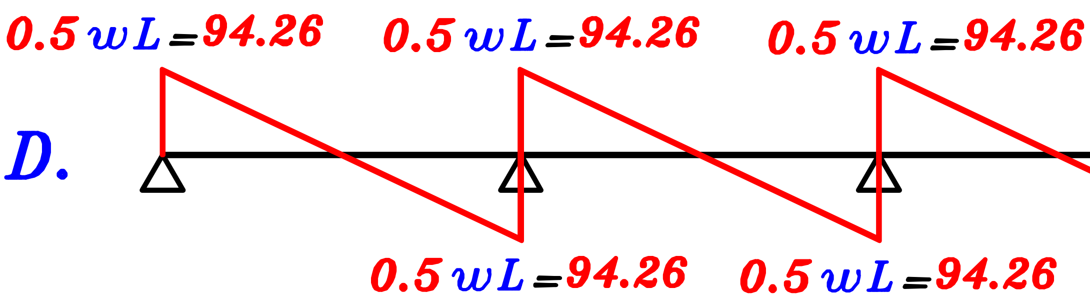
$$M_{t\,(av.)} \simeq \frac{0.3}{L}\,M_o \simeq \frac{0.3}{6.0} * 628.4 = 31.42 \text{ kN.m/m}$$

$$M_{t\,(av.)} = 31.42 \text{ kN.m/m}$$

Torsional moment



T.M.D.



2- Dimensions of Marginal Beam.

Take $b = 400$ mm

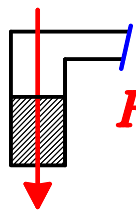
$$* \quad t_{ben.} = 3.50\sqrt{\frac{M}{F_{cu}\,b}} = 3.50\sqrt{\frac{144 * 10^6}{25 * 400}} = 420 \text{ mm}$$

$$* \quad t_{tor.} \simeq \frac{3\,M_t}{1.6 * b^2} = \frac{3 * 94.26 * 10^6}{1.6 * 400^2} = 1104.7 \text{ mm}$$

$$\boxed{t = 1200 \text{ mm}} > 3\,t_s$$

3- Design of Marginal Beam due to Bending.

Sec. (1-1) $(400 * 1200)$ $M_{U.L.} = 144$ kN.m



R-Sec.

$$1100 = C_1\sqrt{\frac{144 * 10^6}{25 * 400}} \longrightarrow C_1 = 9.16 \longrightarrow J = 0.826$$

$$A_s = \frac{144 * 10^6}{0.826 * 360 * 1100} = 440.2 \text{ mm}^2$$

Check $A_{s_{min.}}$ $\quad A_{s_{req.}} = 440.2 \text{ mm}^2$

$$\mu_{min.}\,b\,d = \left(0.225 * \frac{\sqrt{F_{cu}}}{F_y}\right) b\,d = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1100 = 1375 \text{ mm}^2$$

$$\therefore \mu_{min.}\,b\,d > A_{s_{req.}} \xrightarrow{\text{Use}} A_{s_{min.}}$$

$$A_{s_{min.}} = \left(0.225 * \frac{\sqrt{25}}{360}\right) 400 * 1100 = 1375$$

$$1.3\,A_{s\,req.} = (1.3)(440.2) = 572.26$$

الأقل 572.26

st. 360/520 $\quad \frac{0.15}{100}\,b\,d = \frac{0.15}{100}(400)(1100) = 660$

الأكبر 660 mm^2

4- Design the beam For Shear + Torsion.

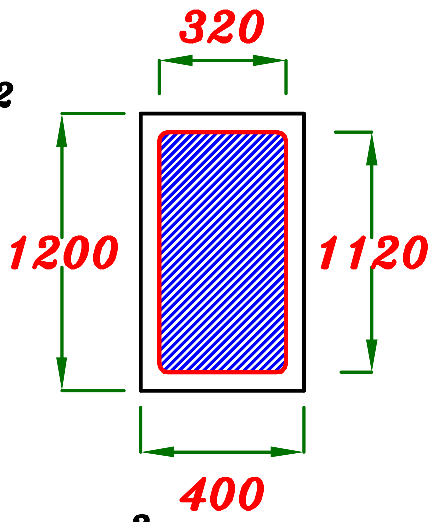
$$q_u = \frac{Q}{b\,d} = \frac{144.0 * 10^3}{400 * 1100} = 0.327 \ N/mm^2$$

$$A_{oh} = 320 * 1120 = 358400 \ mm^2$$

$$A_o = 0.85 * A_{oh} = 0.85 * 358400 = 304640 \ mm^2$$

$$P_h = 2 * 320 + 2 * 1120 = 2880 \ mm$$

$$t_e = \frac{A_{oh}}{P_h} = \frac{358400}{2880} = 124.44 \ mm$$



$$q_{tu} = \frac{M_{tu}}{2\,A_o\,t_e} = \frac{94.26 * 10^6}{2 * 304640 * 124.44} = 1.243 \ N/mm^2$$

$$q_{cu} = (0.24)\sqrt{\frac{25}{1.5}} = 0.98 \ N/mm^2$$

$$q_{t\,min} = (0.06)\sqrt{\frac{25}{1.5}} = 0.245 \ N/mm^2$$

$$q_{u\,max} = (0.7)\sqrt{\frac{25}{1.5}} = 2.85 \ N/mm^2$$

$$\sqrt{q_u^2 + q_{tu}^2} = \sqrt{0.327^2 + 1.243^2} = 1.285 \ N/mm^2 < q_{u\,max} \quad \therefore \text{o.k.}$$

$\boxed{q_u < q_{cu} \ , \ q_{tu} > q_{t\,min}}$ $\therefore$ **Use RFT. For Torsion only.**

* ## Stirrups.

$$\therefore A_{str} = \frac{M_{tu}\ S_t}{(1.7)\ A_{oh}\ \left(\frac{F_y}{\gamma_s}\right)} \qquad \therefore A_{str} = \frac{(94.26 * 10^6) * S_t}{(1.7)(358400)(240/1.15)}$$

$$\therefore \boxed{S_t = 1.349 * A_{str}}$$

* Take $\phi\ 8 \longrightarrow A_{str} = 50.3\ mm^2$

$\therefore S_t = 1.349 * A_{str} = 1.349 * 50.3 = 67.85\ mm < 100\ mm$

* Take $\phi\ 10 \longrightarrow A_{str} = 78.5\ mm^2$

$\therefore S_t = 1.349 * A_{str} = 1.349 * 78.5 = 105.9\ mm > 100\ mm \ \therefore$ o.k.

$$\therefore \text{No. of stirrups} \backslash m' = \frac{1000}{S} = \frac{1000}{105.9} = 9.44 = 10$$

$\therefore$ Use Closed Stirrups $\boxed{10\ \phi 10 \backslash m'}$ 2 branches.

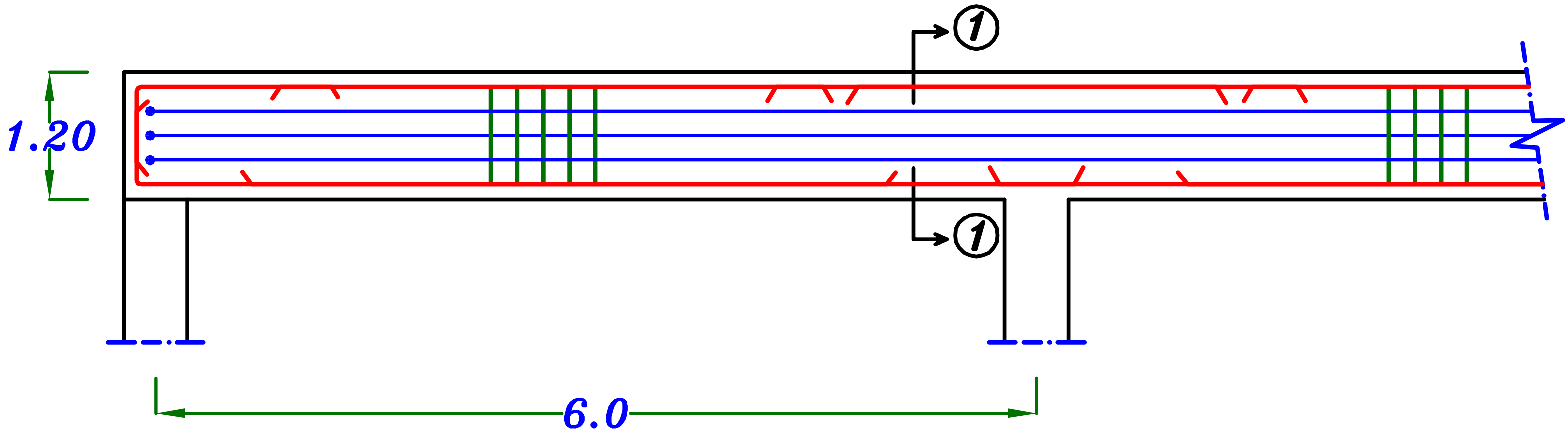
---

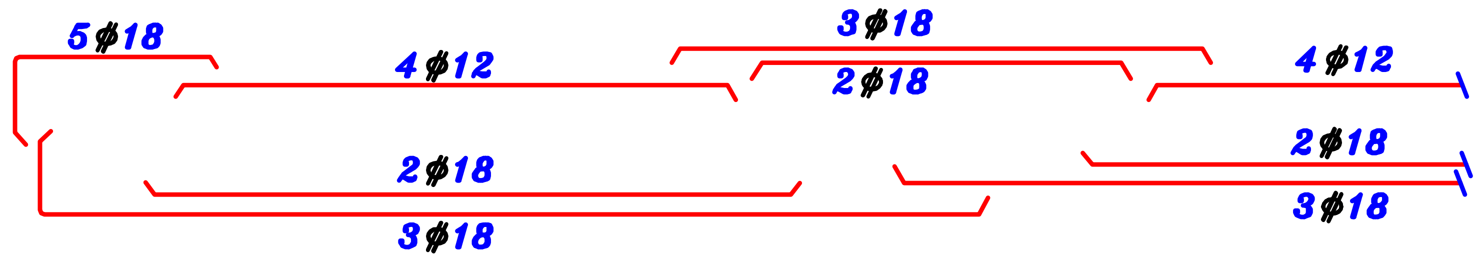
* ## Longitudinal Bars.

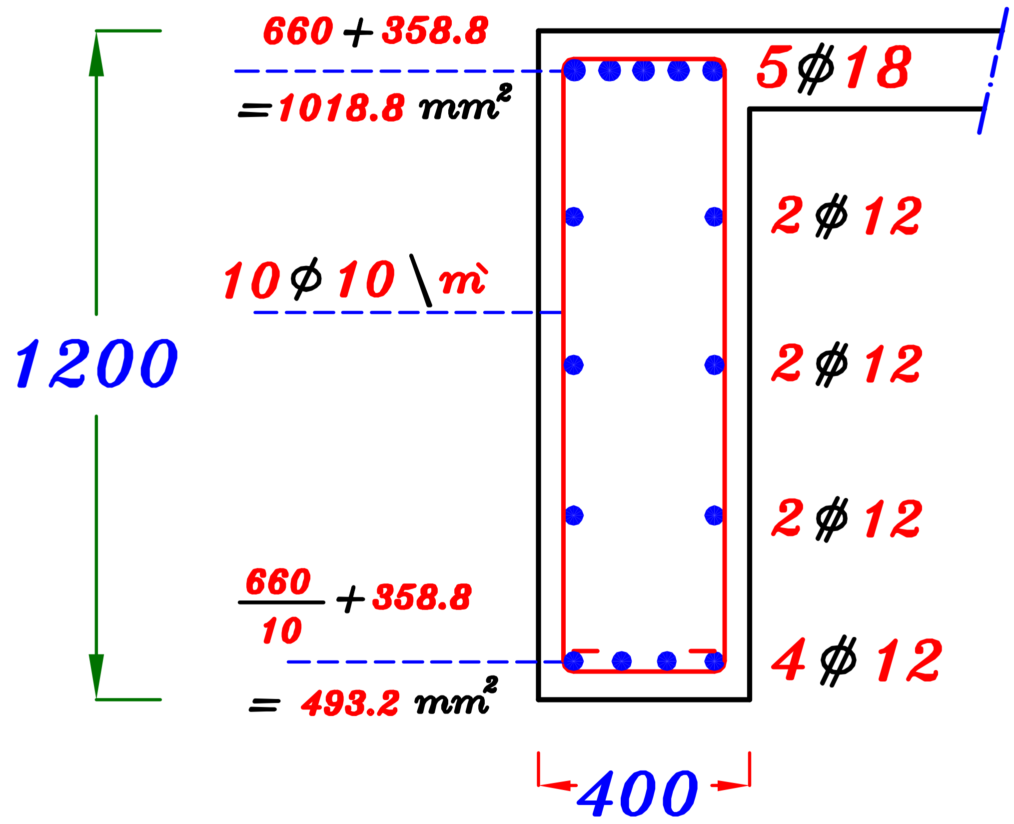
$$S_t = \frac{1000}{10} = 100\ mm$$

$$A_{sl} = \frac{A_{str} * P_h}{S_t}\left(\frac{F_{y\,str.}}{F_{y\,L.b.}}\right) = \frac{(78.5 * 2880)}{100}\left(\frac{240}{360}\right) = 1507.2\ mm^2$$

$$\therefore \frac{A_{sl}}{4} = \frac{1507.2}{4} = 376.8\ mm^2$$


1
1
1.20
6.0

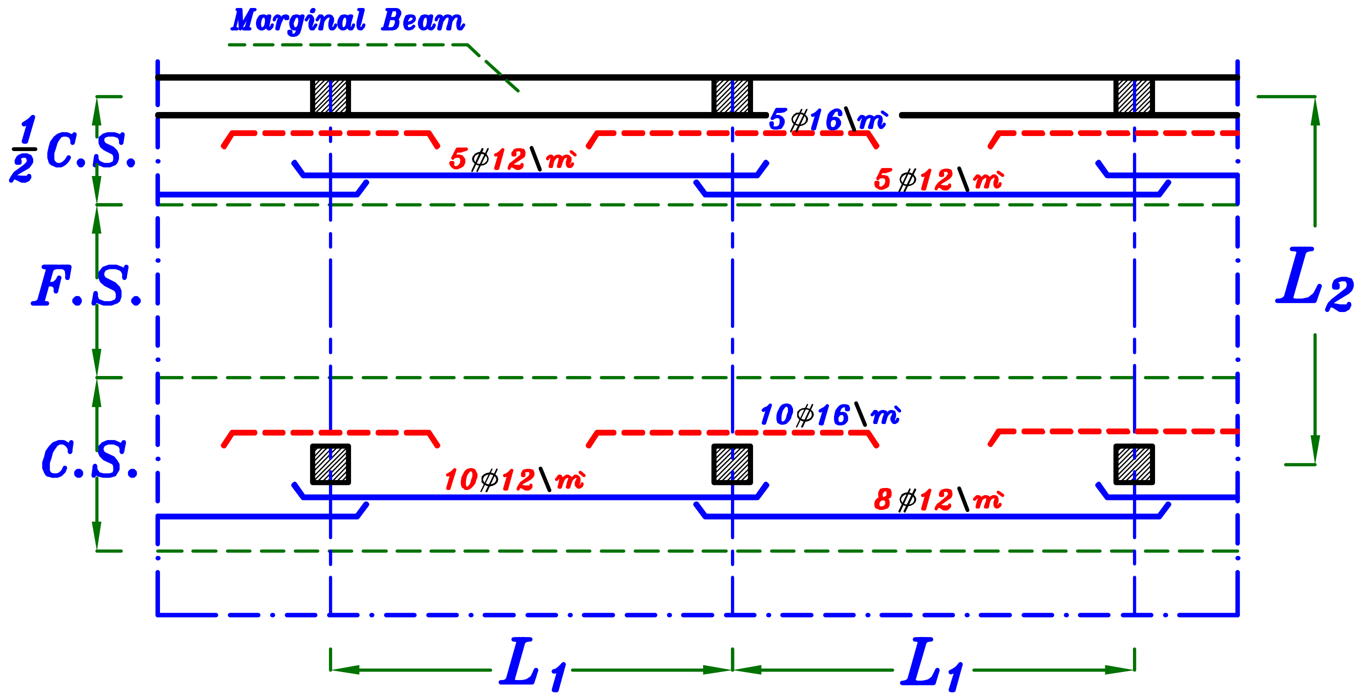

5ϕ18
4ϕ12
3ϕ18
2ϕ18
4ϕ12
2ϕ18
2ϕ18
3ϕ18
3ϕ18


660 + 358.8
= 1018.8 mm²
5ϕ18
2ϕ12
10ϕ10 \m`
1200
2ϕ12
2ϕ12
660/10 + 358.8
= 493.2 mm²
4ϕ12
400

Sec. (1-1)

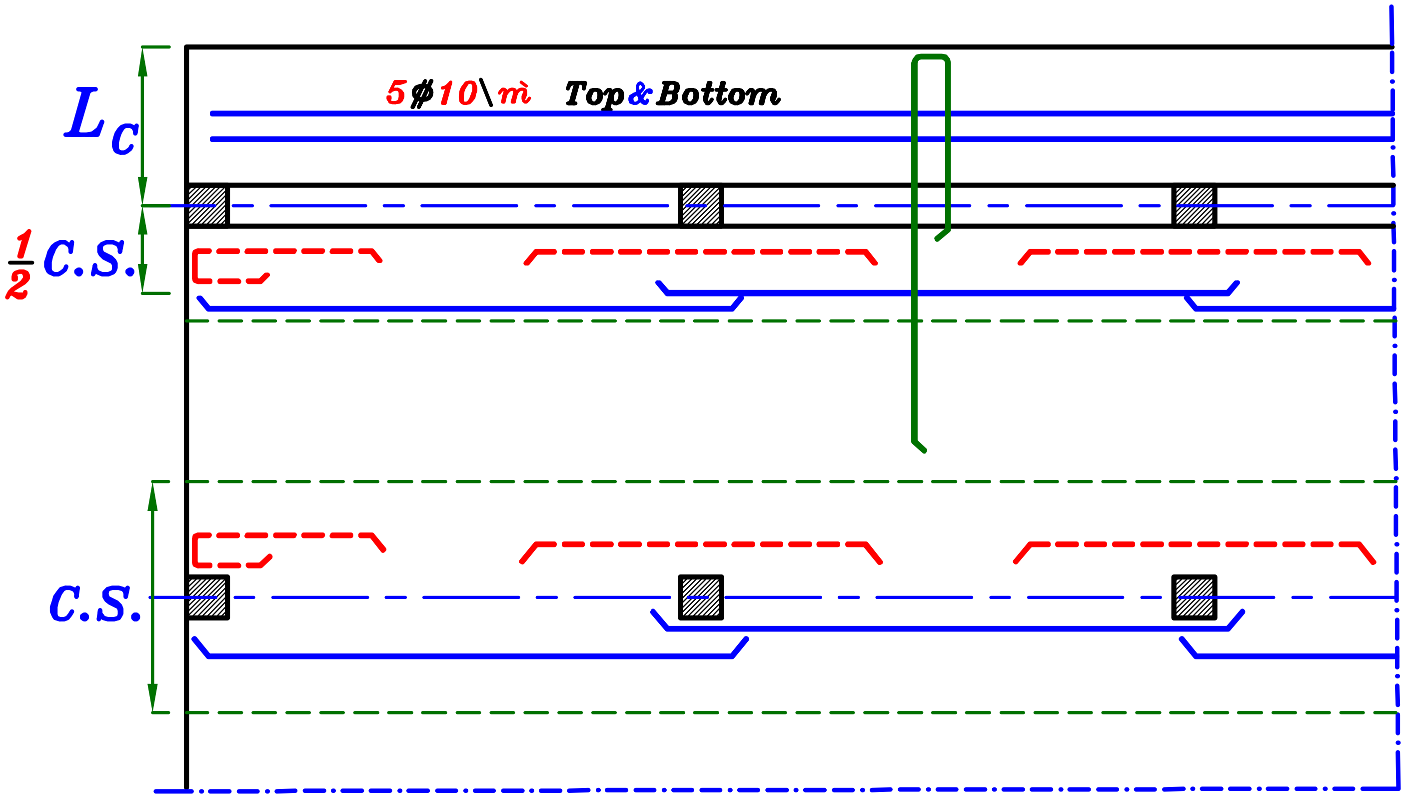
RFT. of Flat Slab with Marginal Beam.

مساحه التسليح الموجود فى ال Column Strip المجاوره لل Marginal Beam
فى المتر الواحد تساوى نصف مساحه الحديد الموجود فى المتر لل Column Strip الرئيسيه



RFT. of Cantilever with Marginal Beam.

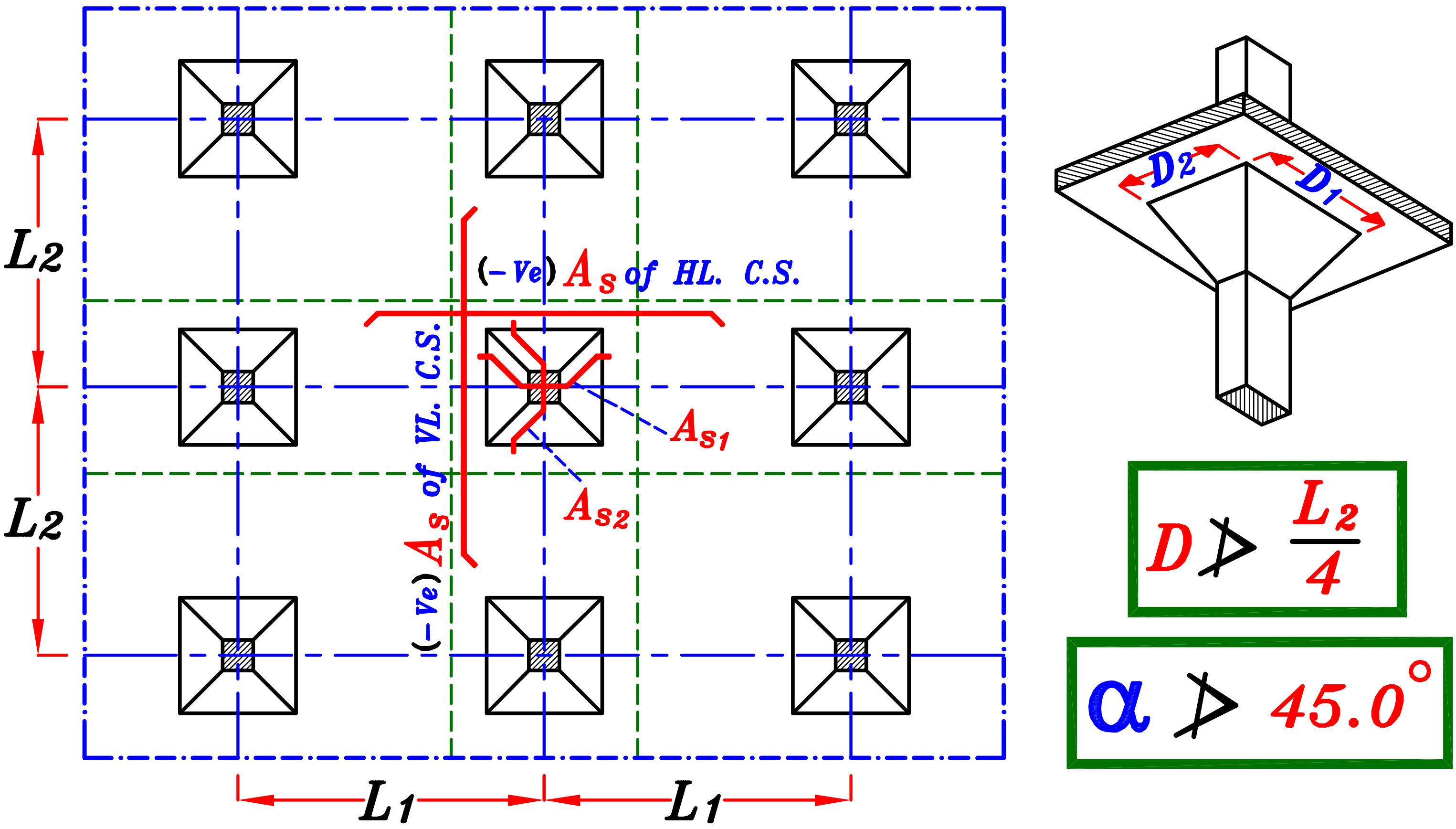
اذا وجدت marginal Beam يعتبر Cantilever Solid Slab
و نضع تسليح عمودى على ال Cantilever عباره عن 5Ø10\m Top & Bottom
لا يعتبر ال Cantilever جزء من أخر C.S. فى الاتجاه العكسى
و يتم وضع تسليح فى اخر شريحه يساوى نصف مساحه الحديد الموجود فى اخر Column Strip الرئيسيه



Column Head.

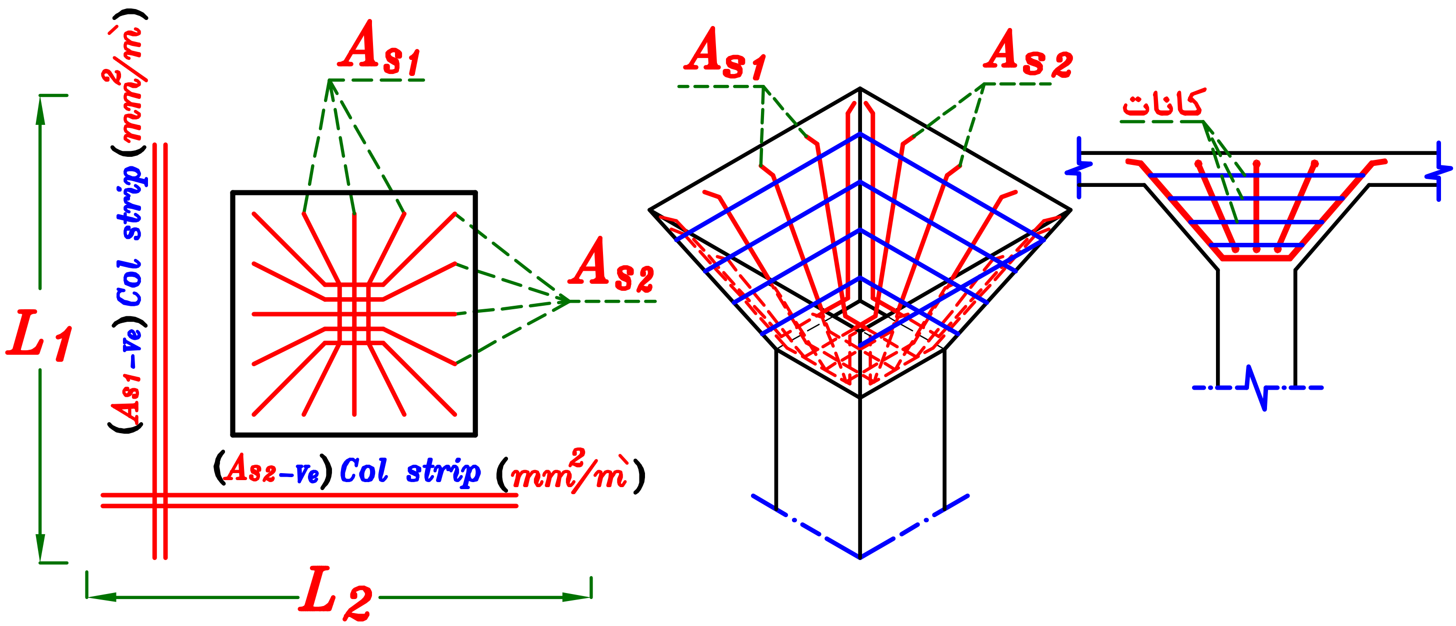
نستخدم ال *Column Head* عندما تكون البلاطه *unsafe punching*.

RFT. of columns Head.



$$D \ngtr \frac{L_2}{4}$$

$$\alpha \ngtr 45.0^{\circ}$$



$$A_{s1} \geqslant 0.04\,(A_{s1-Ve})\ \text{Col Strip}\ (mm^2/m') * L_2$$

$$A_{s2} \geqslant 0.04\,(A_{s2-Ve})\ \text{Col Strip}\ (mm^2/m') * L_1$$

Voids in Flat Slab.

يمكن عمل فتحات (*Voids*) فى ال *Flat Slabs* بأى أبعاد و فى أى مكان

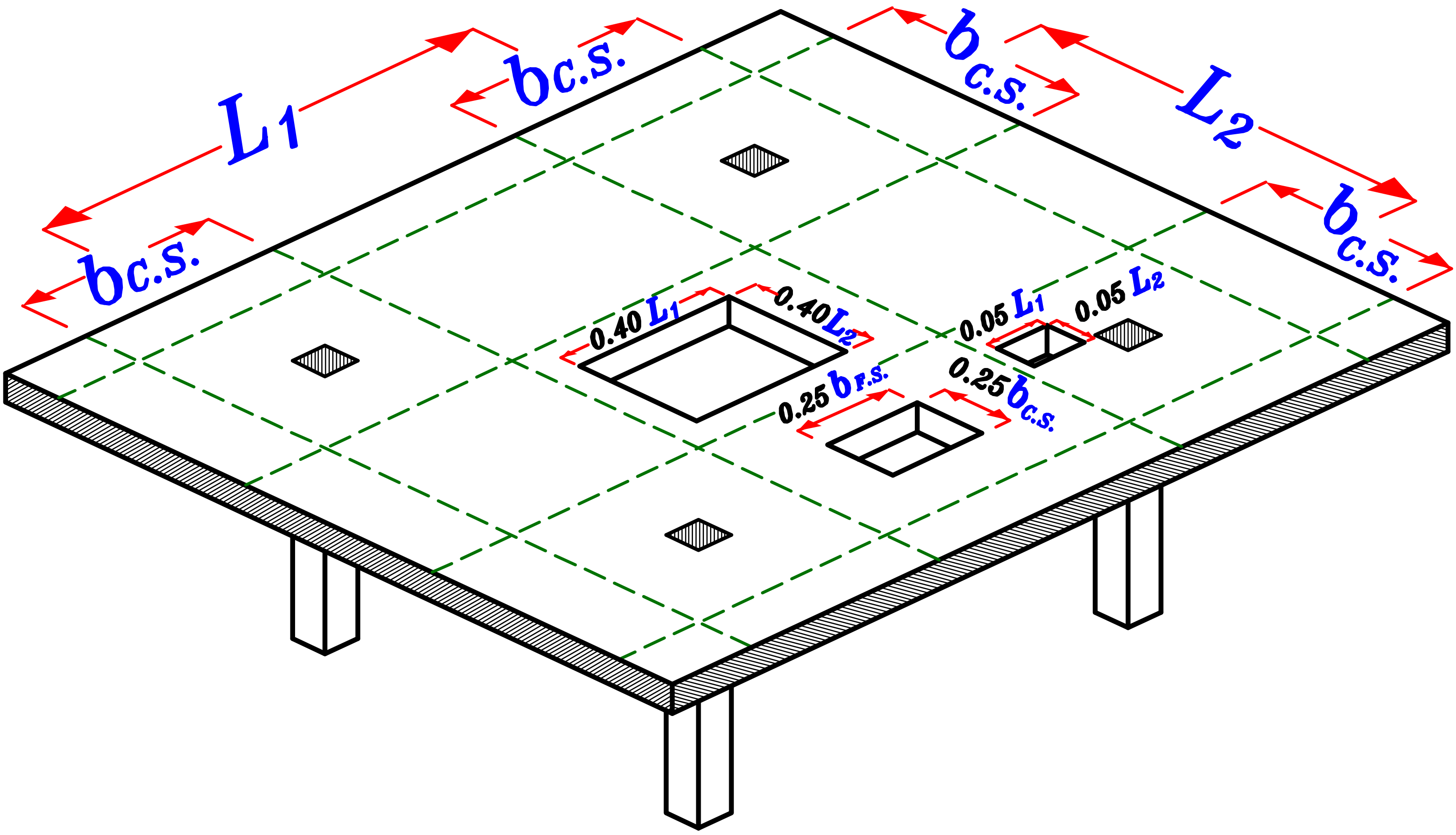
و لكن يجب أن نأخذ تأثير الفتحات عند اجراء التحليل الانشائى للبلاطه

(و ذلك الحل بطريقه الكمبيوتر)

و لكن لكى لا تؤثر ال (*Voids*) فى قيم عزوم البلاطات عند الحل ب

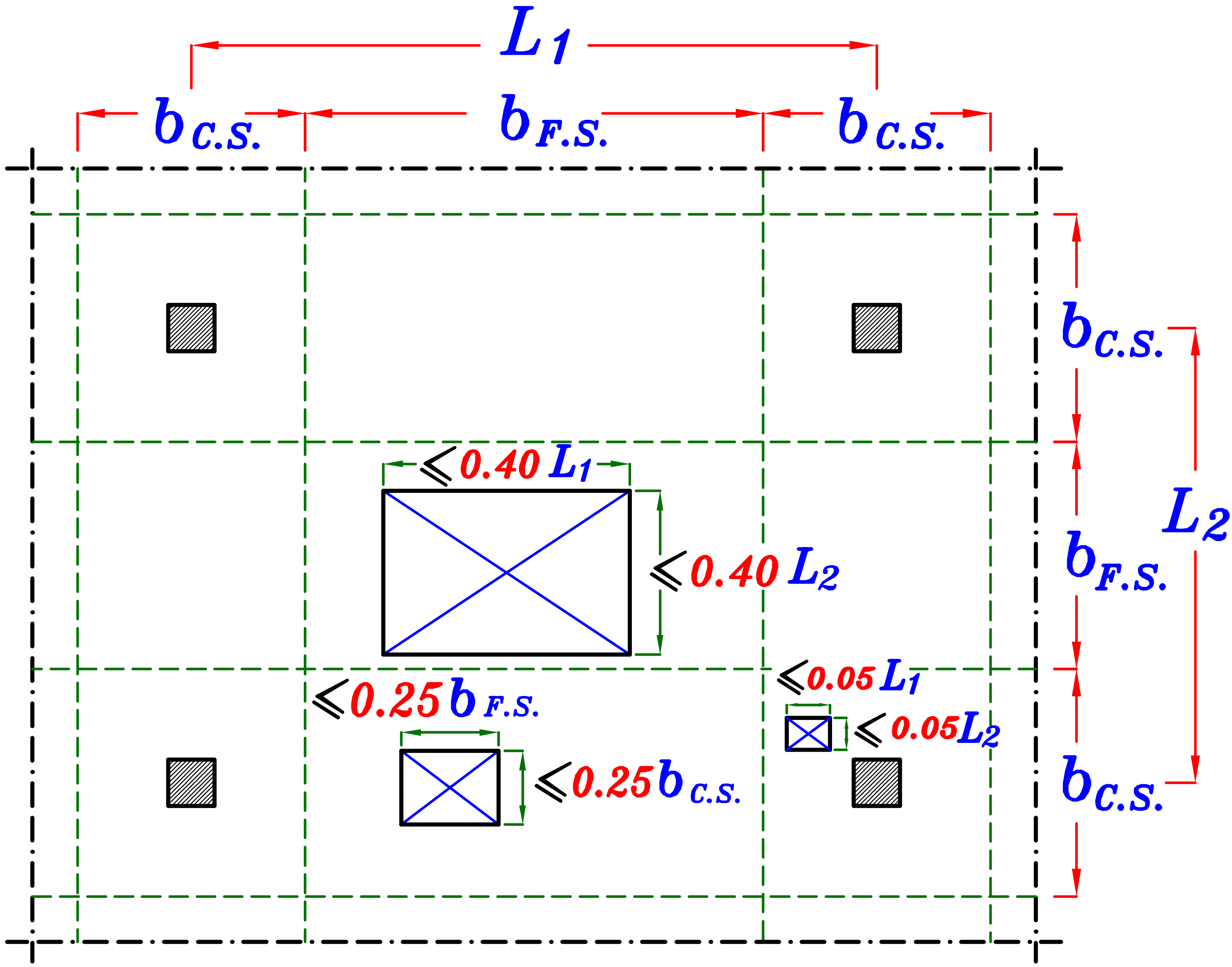
(*Empirical method or Frame analysis method*)

يجب أن لا تتعدى أبعاد الفتحات للقيم الاتيه :



max. dimensions of Voids in Flat slab

IF it hasn't been solved by computer.



الابعاد المسموح بها فى فتحات ال Flat Slab

١- تقاطع F.S. مع F.S.

max. length at L_1 Direction $\not> 0.4\, L_1$

max. length at L_2 Direction $\not> 0.4\, L_2$

٢- تقاطع F.S. مع C.S.

max. length at Column strip $\not> 0.25\, b_{C.S.}$

max. length at Field strip $\not> 0.25\, b_{F.S.}$

٣- تقاطع C.S. مع C.S.

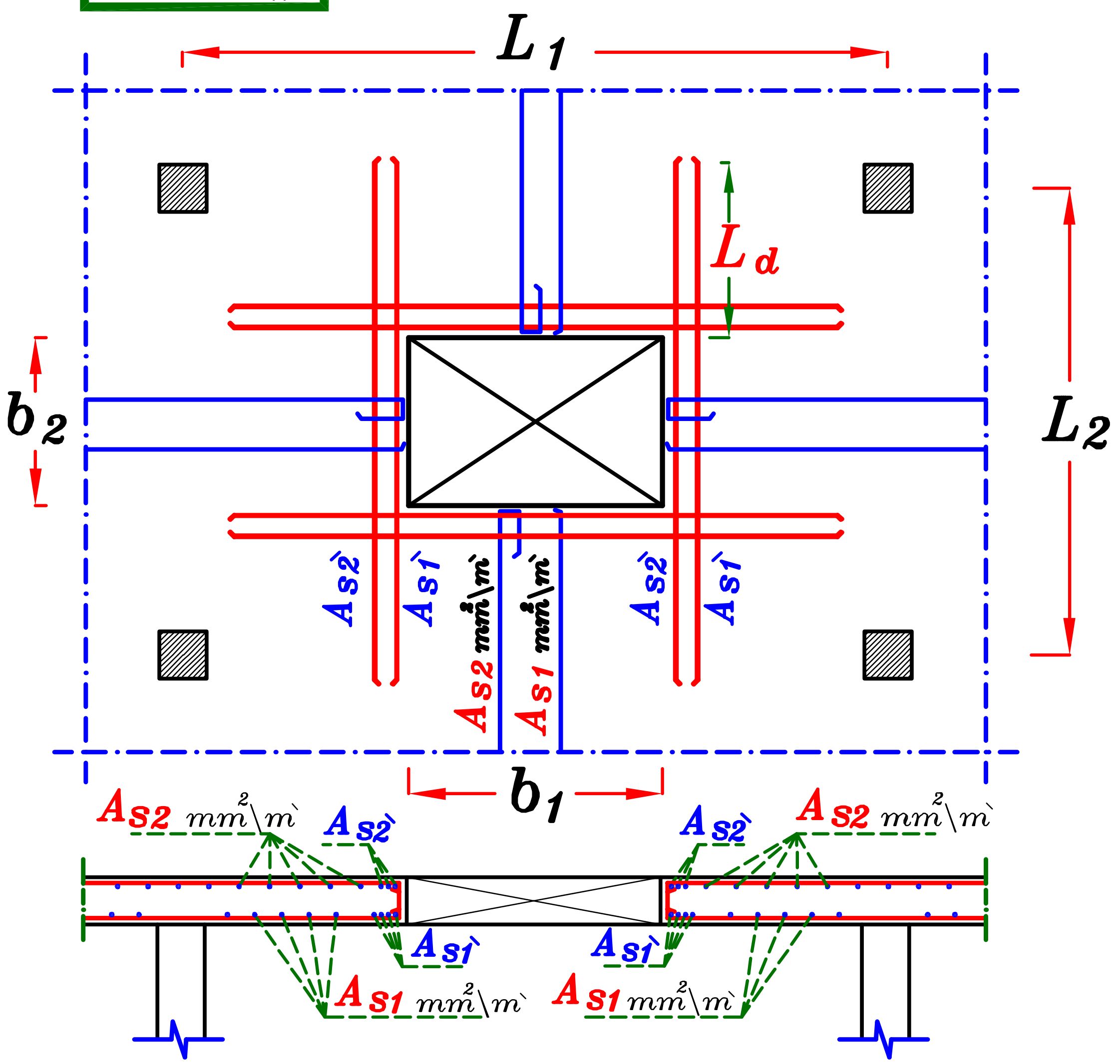
max. length at L_1 Direction $\not> 0.05\, L_1$

max. length at L_2 Direction $\not> 0.05\, L_2$

تسليح البلاطات عند الفتحات. RFT. of Flat Slab at the Opening.

يتم حساب مساحه التسليح المقطوع بواسطه الفتحه (علوى و سفلى)

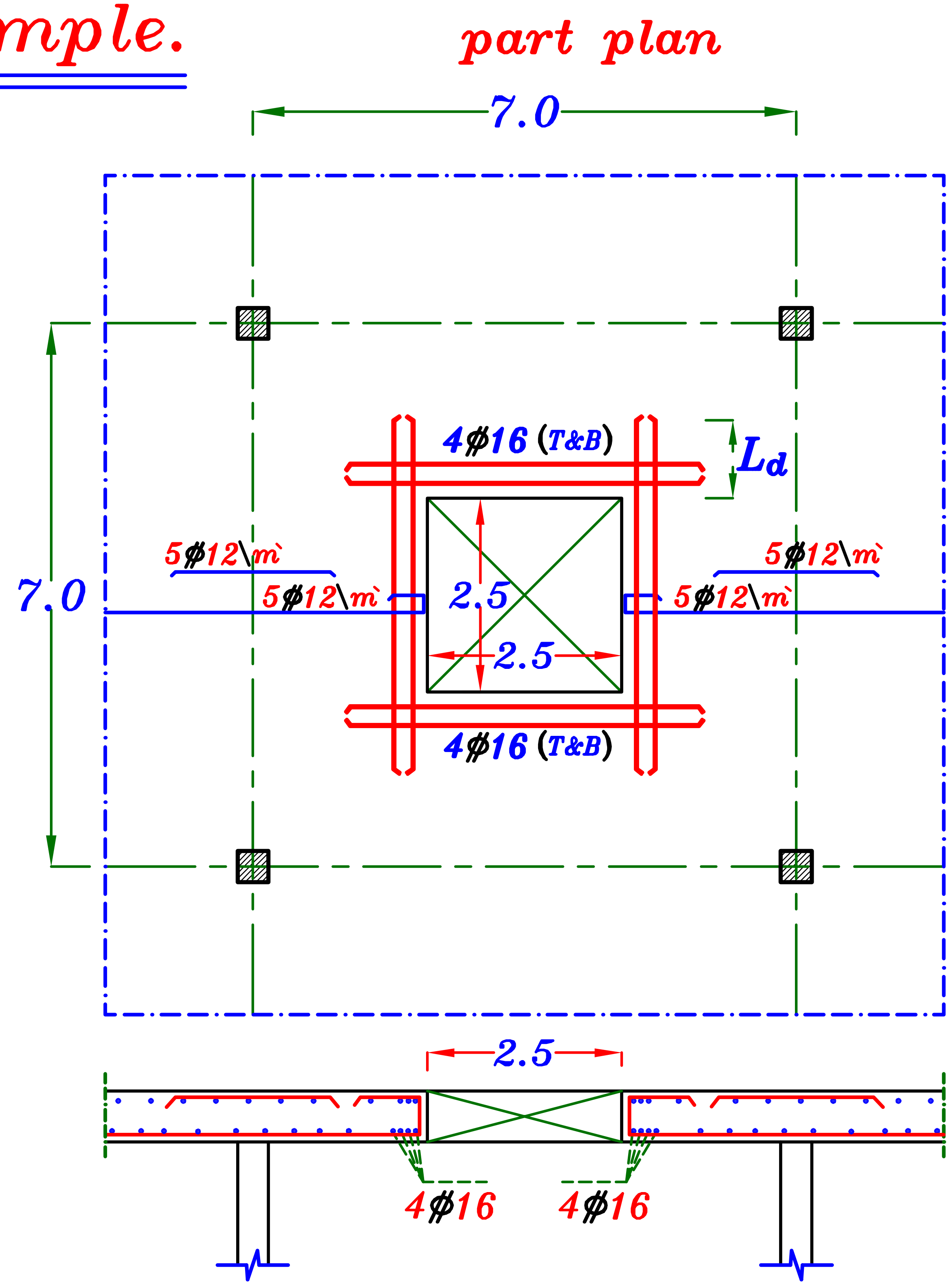
و يتم تركيزه على جانبى الفتحه و يمتد بعد الفتحه مسافه $\boxed{L_d = 60\,\phi}$



$$A_{s1`} = \frac{1}{2}\left[A_{s1}\,(mm^2\backslash m`) * b_1\,(m)\right] = \checkmark\ mm^2$$

$$A_{s2`} = \frac{1}{2}\left[A_{s2}\,(mm^2\backslash m`) * b_2\,(m)\right] = \checkmark\ mm^2$$

Example.



A_s /m` = 5ϕ12 = 5 * 113 = 565 mm^2

A_s (المقطوع بواسطه الفتحه) = 565 * 2.5 m = 1412.5 mm^2

A_s (each side) = $\frac{1412.5}{2}$ = 706.2 mm^2 $\boxed{4ϕ16}$

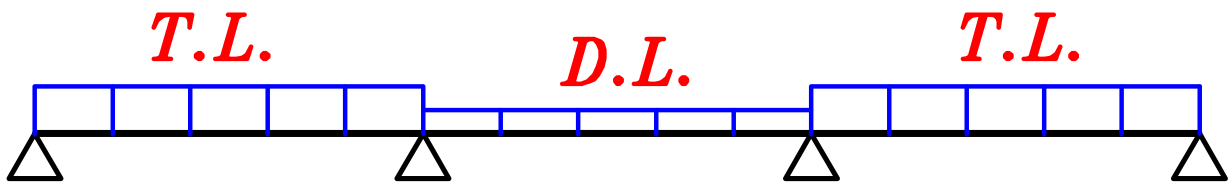
A_s (each side) = $\boxed{4ϕ16}$ (T&B)

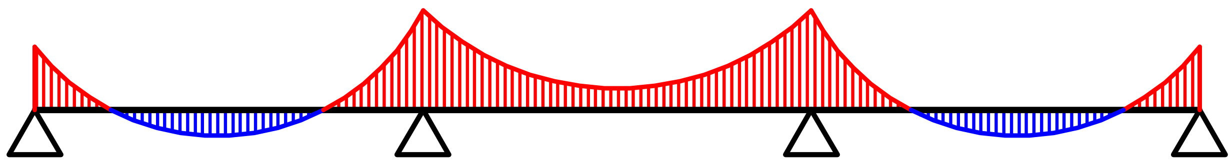
Case of Heavy Live Loads.

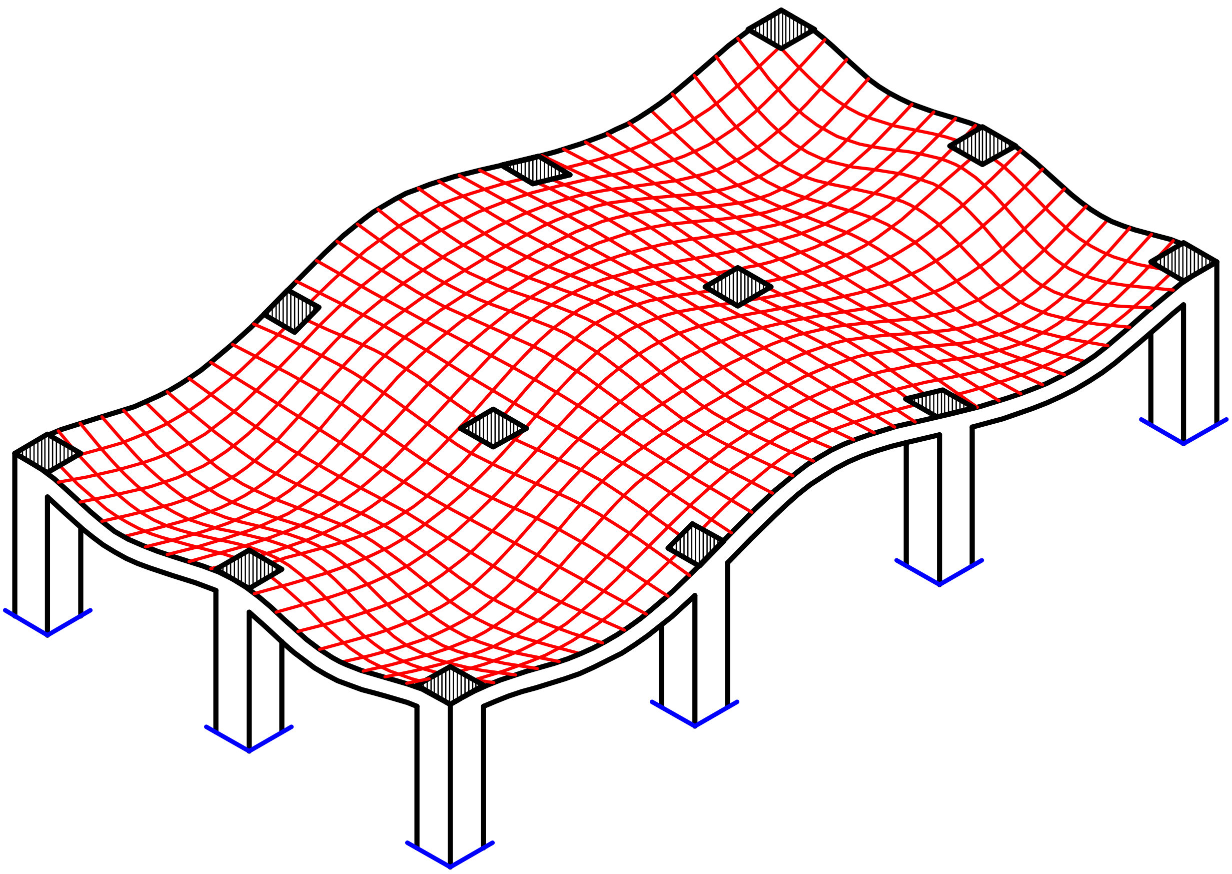
في حاله ($L.L. > 1.5\ D.L.$)

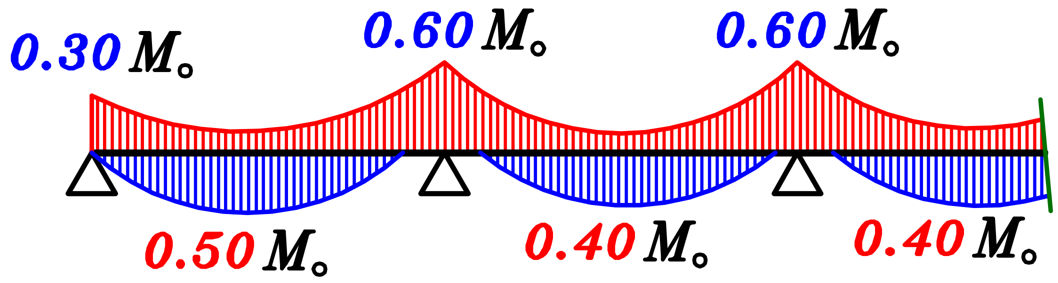
أى أن $\boxed{L.L. > 1.5\left[t_s \gamma_c + F.C. + Wall\right]}$

ممكن من حالات التحميل ان يتكون *moments* ($-Ve$) على كل الباكيه ناتج من حالات التحميل .





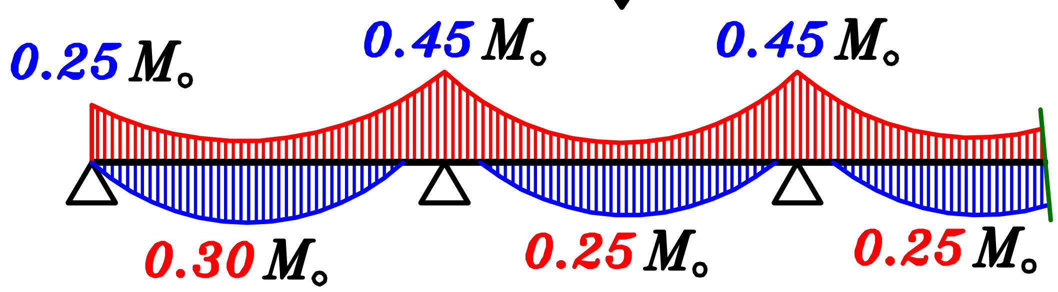




max-max B.M.D.

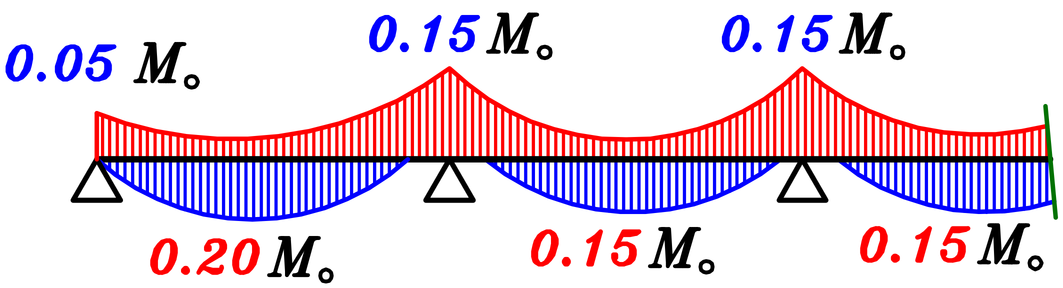
على كل الباكيه

من C.L. الى C.L.



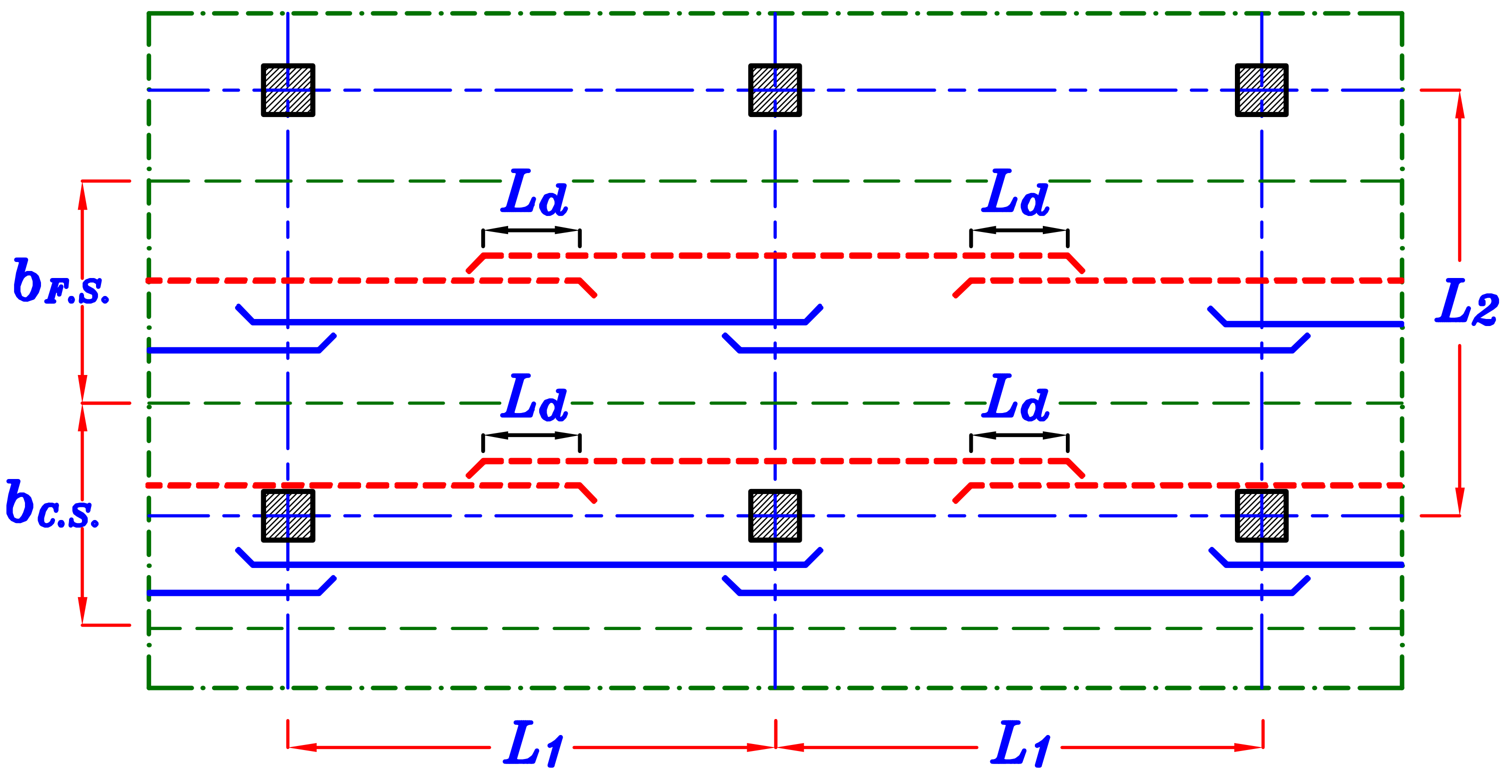
Column Strip

$(M_{c.s.})$



Field Strip

$(M_{F.S.})$



يتم عمل وصلات فى الحديد العوى عند منتصف الـ **span** بطول $\boxed{L_d = 60\,\phi}$

Design of Columns of Flat Slab.

تصمم أعمده الـ Flat Slab على Moment & Normal (M, P)

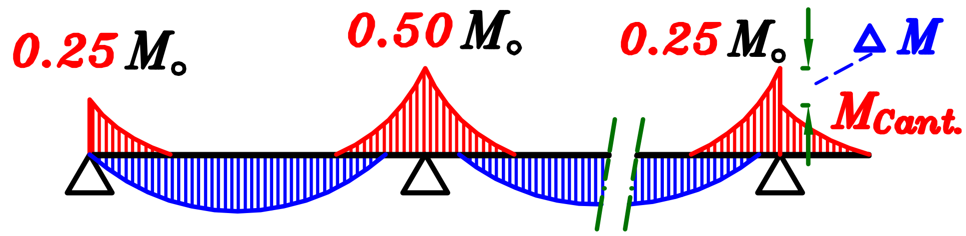
عند تصميم الـ Flat Slab بطريقه Empirical method تحسب العزوم على الاعمده نسبه من عزوم البلاطه كما سنرى لاحقا .

و لكن عند تصميم الـ Flat Slab بطريقه Frame analysis يؤخذ العزم على الاعمده كما هو محسوب فى الـ Frame

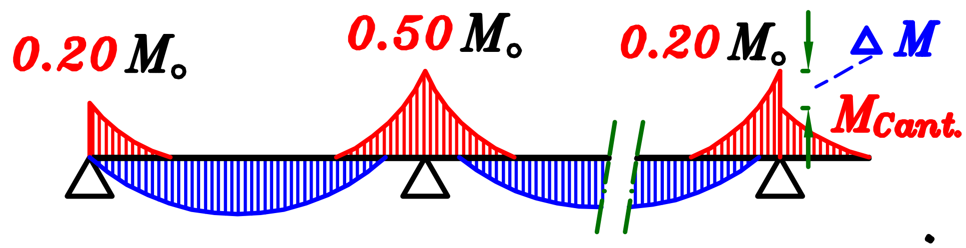
Solving using Empirical method.

حيث تحسب الـ (P) من وزن البلاطه التى يحملها العمود مضافاً اليها وزن العمود نفسه و كل ذلك مضروباً فى عدد الادوار التى يحملها العمود .

و تحسب الـ (M) كنسبه من الـ moments $(-Ve)$ الموجوده على الـ Column Strip .



C.S.
Without
Marginal Beam.



C.S.
With
Marginal Beam.

و تكون النسب كالتالى :

1. Interior Column. عمود داخلى $\longrightarrow$ $50\%\ M_{C.S.}$
2. Edge Column. عمود طرفى $\longrightarrow$ $90\%\ M_{C.S.}$
3. Column at cantilever. عمود عند الكابولى $\longrightarrow$ $90\%\ \Delta M$

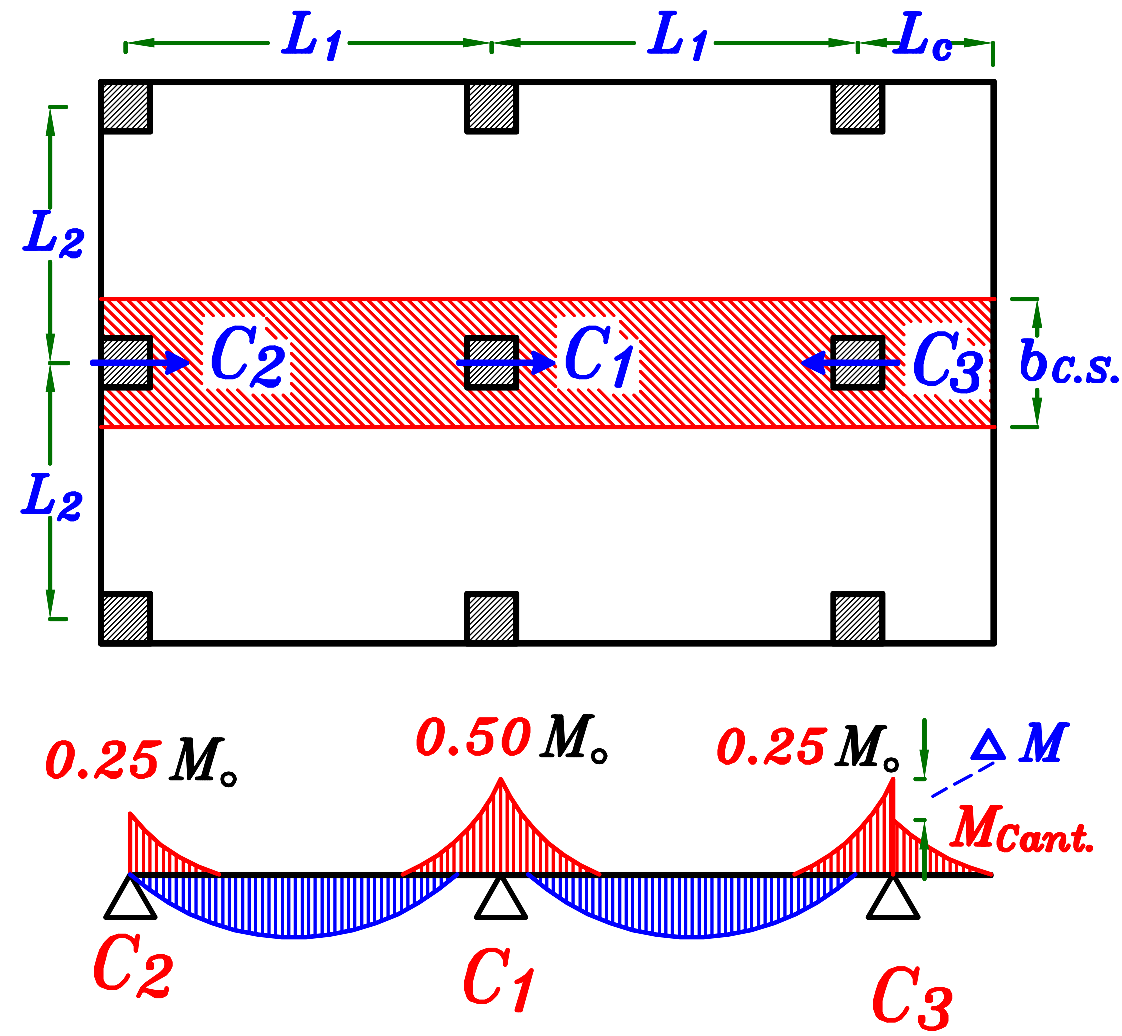
   حيث ΔM هى الفرق بين (Edge moment – Cantilever moment)
4. Corner Column. عمود ركنى $\longrightarrow$ $90\%\ M_{C.S.} * 0.5$

يتم ضرب العزم فى 0.5 لان الشريحه الطرفيه تحمل نصف أحمال الشريحه الوسطيه .

نسب توزيع عزوم ال Column Strip على الاعمده كل حسب مكانه.

لحساب أكبر moment على أعمده C_1, C_2, C_3

يكون نسبه من moments (−Ve) للشريحه فى الاتجاه الطويل .



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① **Interior Column.** C_1

$$M_C = 50\%\ M_{c.s.} = 0.5\,(0.50\,M_o)$$

② **Edge Column.** C_2

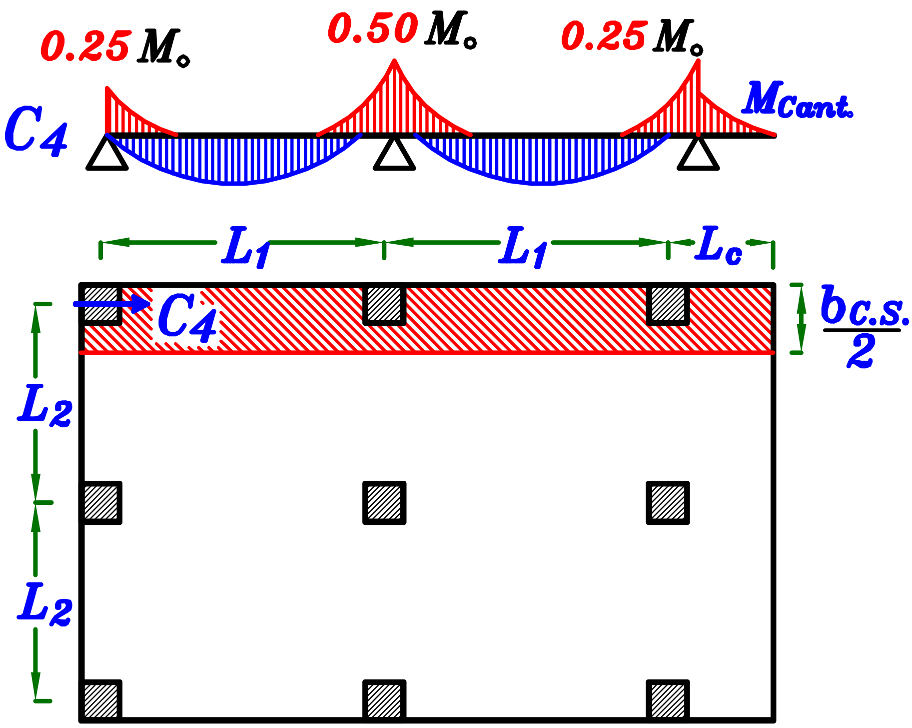
$$M_C = 90\%\ M_{c.s.} = 0.9\,(0.25\,M_o)$$

③ **Column at cantilever.** C_3

$$M_C = 90\%\ \triangle M = 0.9\,(0.25\,M_o - M_{Cant.})$$

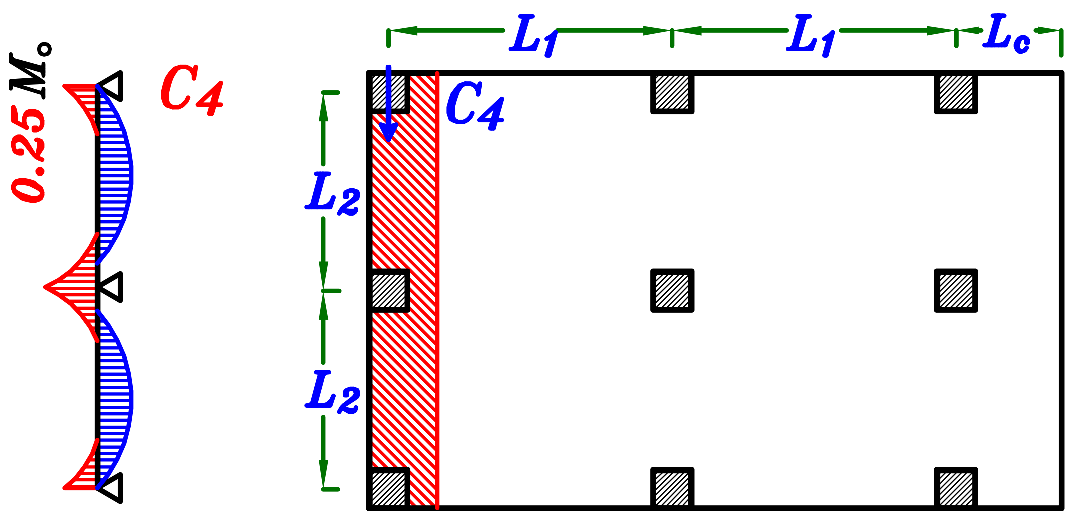
لحساب أكبر moment على العمود C_4

يكون نسبه من moments $(-Ve)$ للشريحه فى الاتجاهين .



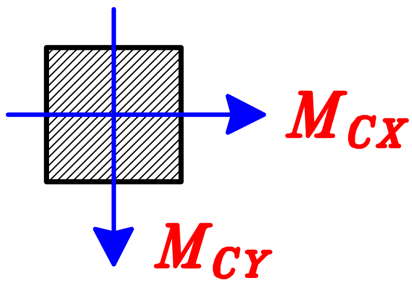
$$M_{CX} = 90\% \, M_{c.s.} * 0.5 = 0.9 \, (0.25 \, M_\circ) * 0.5$$

يتم ضرب العزم فى 0.5 لان الشريحه الطرفيه تحمل نصف أحمال الشريحه الوسطيه .

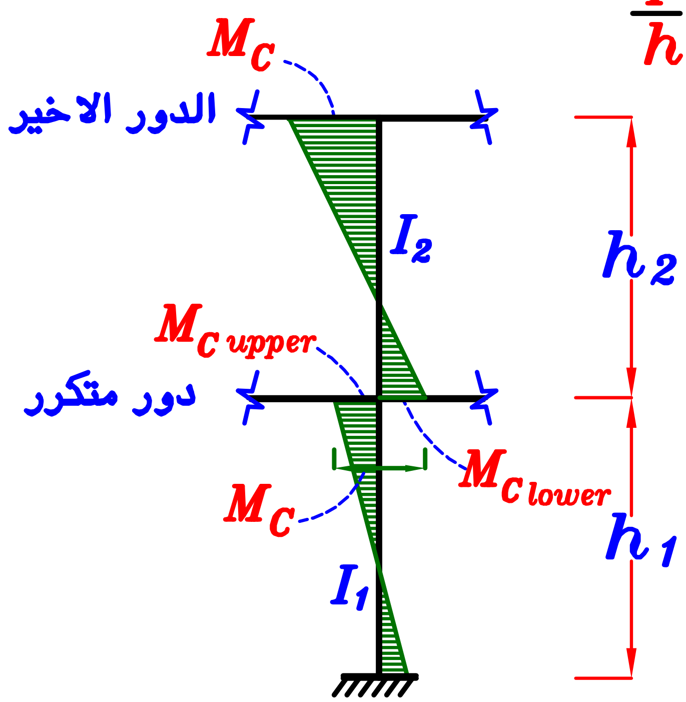


$$M_{CY} = 90\% \, M_{c.s.} * 0.5 = 0.9 \, (0.25 \, M_\circ) * 0.5$$

يتم تصميم العمود على M_{CX} , M_{CY}



يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$ فى الادوار المتكرره



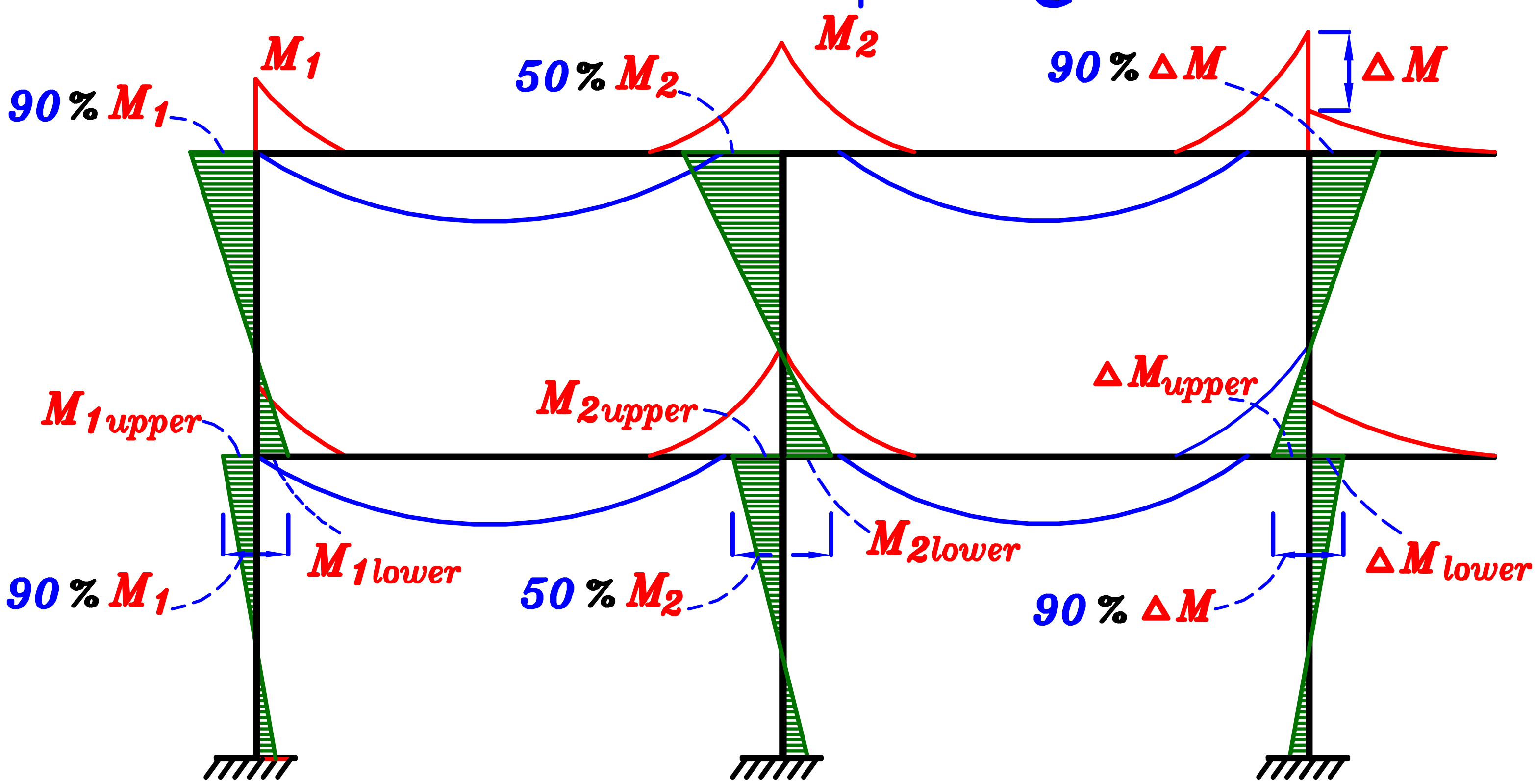
$$\boxed{\sum I\backslash h = I_1\backslash h_1 + I_2\backslash h_2}$$

$$\boxed{M_{C_{lower}} = \frac{(I_1\backslash h_1)}{\sum I\backslash h} * M_C}$$

$$\boxed{M_{C_{upper}} = \frac{(I_2\backslash h_2)}{\sum I\backslash h} * M_C}$$

ما عدا عمود الدور الاخير يأخذ العزم M_C كله لانه لا يوجد عمود علوى .

شكل توزيع العزوم على اعمده الـ Flat Slab



العزوم الموجوده على الاعمده تكون ناتجه من عمل حالات تحميل للبلاطه .
أى عند تصميم الاعمده على هذه العزوم يتم أخذ الـ Normal على العمود من نفس حالة التحميل .

$$g_S = 0.9\left[t_s \gamma_c + F.C. + walls\right]$$

$$w_S = 1.4\left[t_s \gamma_c + F.C. + walls\right] + 1.6\,(L.L.)$$

① Interior Column. عمود داخلى

توجد للعمود الداخلى حالتان تحميل تجعله critical

Case ① Total Load (Critical Case For Ground Floor).

يحمل العمود مساحه من C.L. الى C.L. $(L_1 * L_2)$

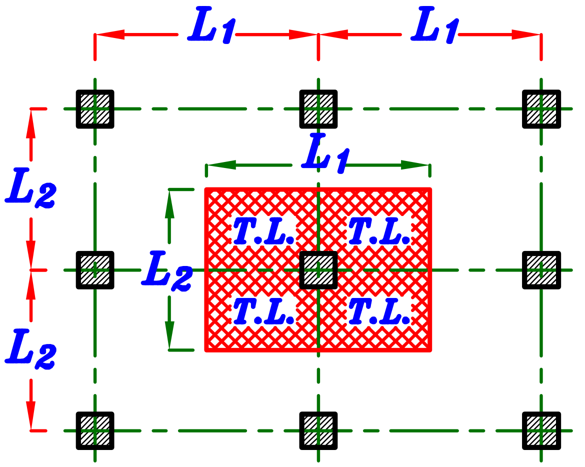
$$P\backslash Floor = w_S * (L_1 * L_2) * 1.1$$

due to own weight of column

$$P_{(total)} = (P\backslash Floor) * No.\ of\ Floors$$

$$M_{ext.} = Zero$$

Designed on $P,\ M_{add}$ ----- due to buckling



Case ② max moment (Critical Case For Last Floor).

يحمل العمود مساحه من C.L. الى C.L. $(L_1 * L_2)$

$$P\backslash Floor = \left[w_S * \left(\frac{L_1}{2} * L_2\right) + g_S * \left(\frac{L_1}{2} * L_2\right)\right] * 1.1$$

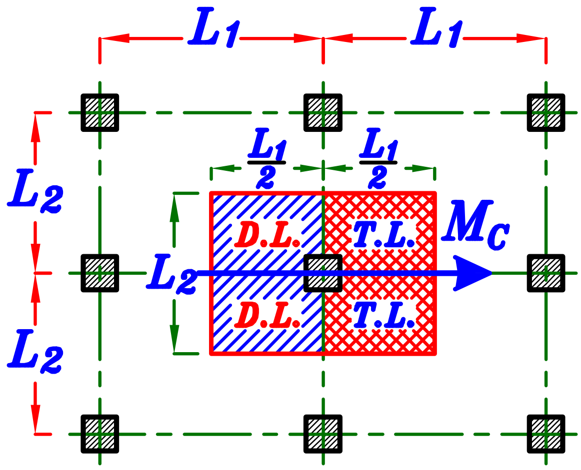
due to own weight of column

$P_{(total)} = (P\backslash Floor) * 1.0$ لانه أخر دور

$$M_C = 50\,\%\, M_{C.S.} = 0.5 * (0.50\, M_o)$$

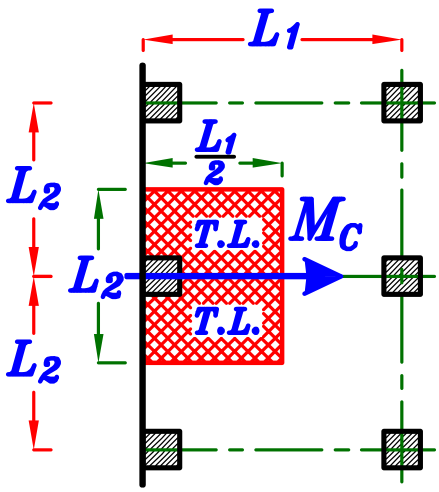
و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

Designed on $P,\ M_{ext},\ M_{add}$ ----- due to buckling



② Edge Column. عمود طرفى

توجد للعمود الطرفى حاله تحميل واحده تجعله critical



يحمل العمود مساحه من C.L. الى C.L. $\left(\frac{L_1}{2} * L_2\right)$

$$P\backslash Floor = w_s * \left(\frac{L_1}{2} * L_2\right) * 1.1$$

due to own weight of column

$$P_{(total)} = (P\backslash Floor) * \text{No. of Floors}$$

$$M_C = 90\,\%\, M_{c.s.} = 0.9 * (0.25\, M_o) \quad \text{Without marginal beam}$$

$$M_C = 90\,\%\, M_{c.s.} = 0.9 * (0.20\, M_o) \quad \text{With marginal beam}$$

و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

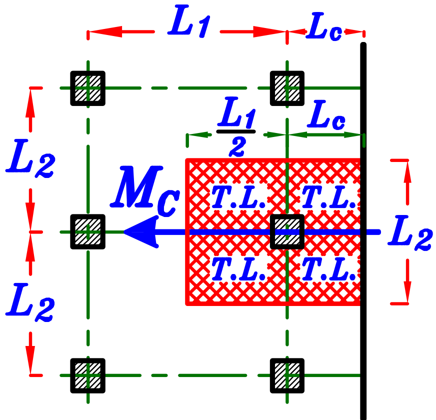
Designed on $P,\ M_{ext},\ M_{add}$ ----- due to buckling

③ Column at cantilever. عمود عند الكابولى

توجد للعمود عند الكابولى حالتان تحميل تجعله critical

Case ① Total Load (Critical Case For Ground Floor).

يحمل العمود مساحه $\left(\frac{L_1}{2}+L_c\right)*L_2$



$$P\backslash Floor = w_s*\left[\left(\frac{L_1}{2}+L_c\right)*L_2\right]*1.1$$

due to own weight of column

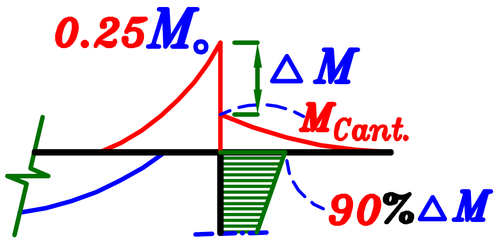
$$P_{(total)} = (P\backslash Floor)*No.\ of\ Floors$$

$$\boxed{M_{Cant.(T.L.)} = \frac{w_s*L_c^2}{2}*b_{c.s.}}$$

عزم ال Cantilever لل Column Strip

$$M_C = 90\%\,(\triangle M)$$

$$= 0.9*\left[0.25\,M_o - M_{Cant.(T.L.)}\right]$$

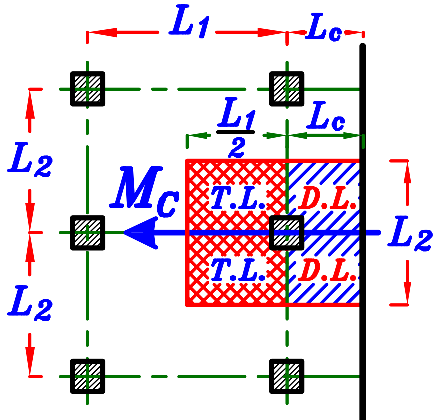


و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

Designed on P, M_{ext}, M_{add} ---- due to buckling

Case ② max moment (Critical Case For Last Floor).

يحمل العمود مساحه $\left(\frac{L_1}{2}+L_c\right)*L_2$



$$P\backslash Floor = \left[w_s\left(\frac{L_1}{2}*L_2\right)+g_s\left(L_c*L_2\right)\right]*1.1$$

due to own weight of column

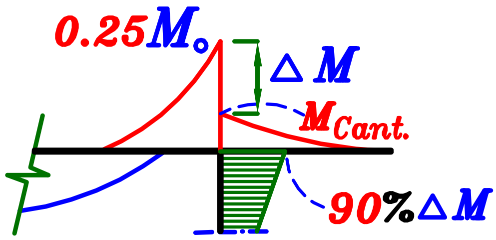
$$P_{(total)} = (P\backslash Floor)*1.0$$ لانه أخر دور

$$\boxed{M_{Cant.(D.L.)} = \frac{g_s*L_c^2}{2}*b_{c.s.}}$$

عزم ال Cantilever لل Column Strip

$$M_C = 90\%\,(\triangle M)$$

$$= 0.9*\left[0.25\,M_o - M_{Cant.(D.L.)}\right]$$



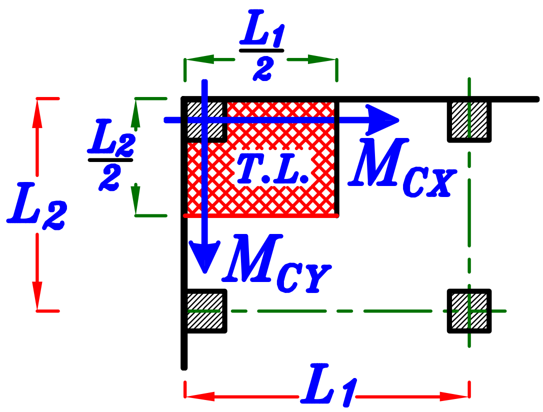
و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

Designed on P, M_{ext}, M_{add} ---- due to buckling

④ Corner Column. عمود ركنى

توجد للعمود الركنى حاله تحميل واحده تجعله critical

يحمل العمود مساحه من C.L. الى C.L. $\left(\frac{L_1}{2} * \frac{L_2}{2}\right)$



$$P\backslash Floor = w_s * \left(\frac{L_1}{2} * \frac{L_2}{2}\right) * 1.1$$

due to own weight of column

$$P_{(total)} = (P\backslash Floor) * No.\ of\ Floors$$

$$M_{CX} = 90\ \%\ M_{C.S.} * 0.5 = 0.9 * (0.25\ M_o) * 0.5 \quad \text{For H.L. Strip}$$

$$M_{CY} = 90\ \%\ M_{C.S.} * 0.5 = 0.9 * (0.25\ M_o) * 0.5 \quad \text{For V.L. Strip}$$

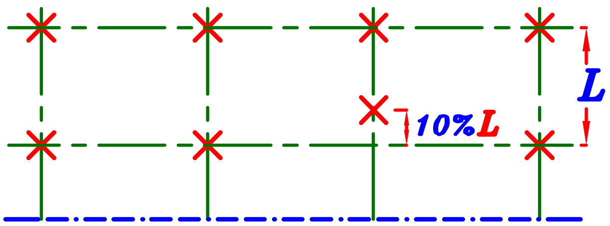
يتم ضرب العزم فى 0.5 لان الشريحه الطرفيه تحمل نصف أحمال الشريحه الوسطيه .

و يوزع العزم M_{CY} , M_{CX} على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

Designed on P, $M_{X\,ext}$, $M_{Y\,ext}$, M_{add}

② Solving Flat Slabs using Frame Analysis Method.

يوجد شرط واحد فقط لامكانيه الحل بهذه الطريقه .



و هو أن تكون الأعمده فى خطوط مستقيمه

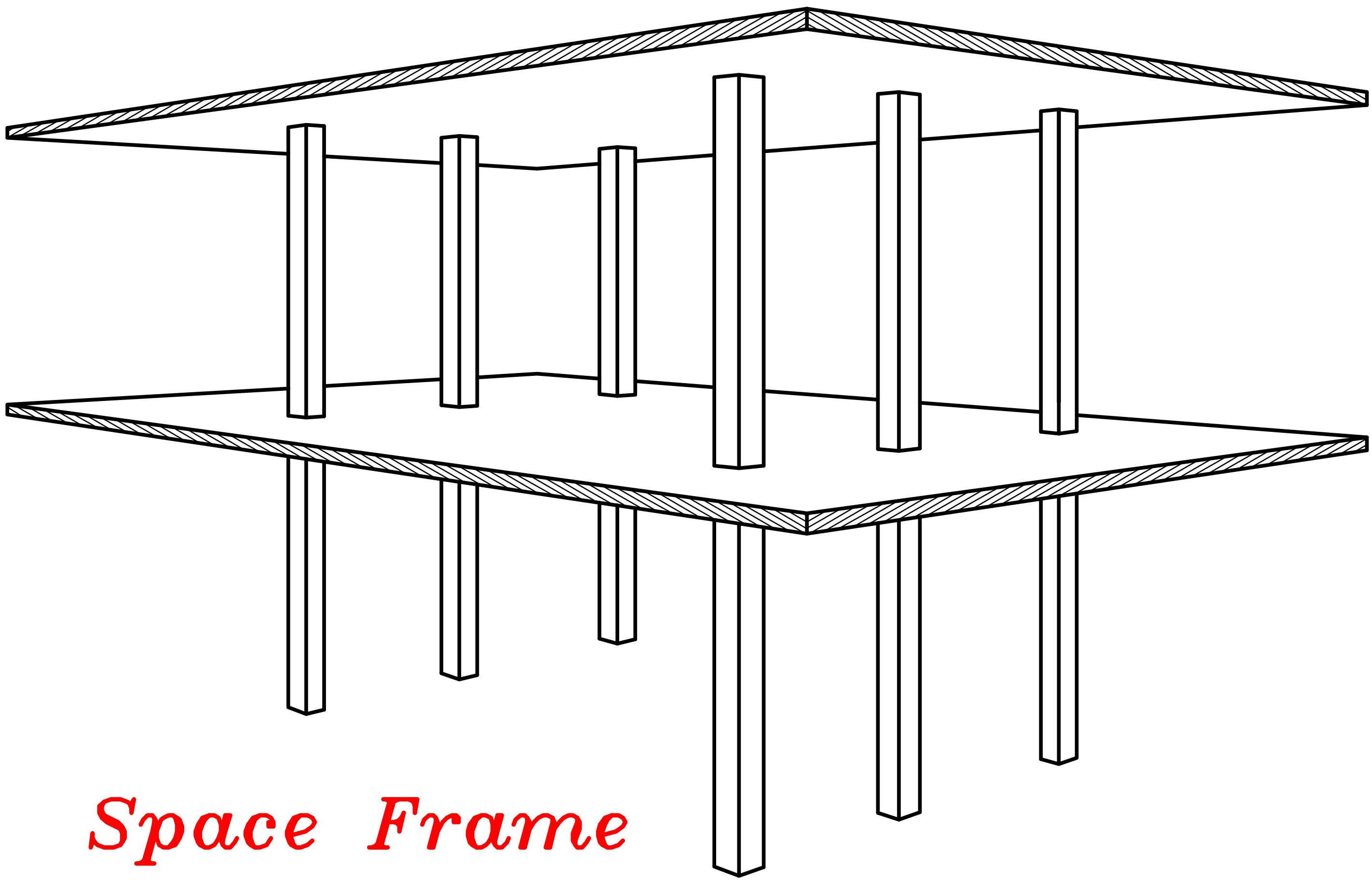
أو بتفاوت لا يزيد عن ١٠ ٪ من طول الباكيه .

و اذا زادت النسبه عن ١٠ ٪ يجب ان نحل البلاطه بالكمبيوتر .

و تعتمد هذه الطريقة على عده إعتبارات :-

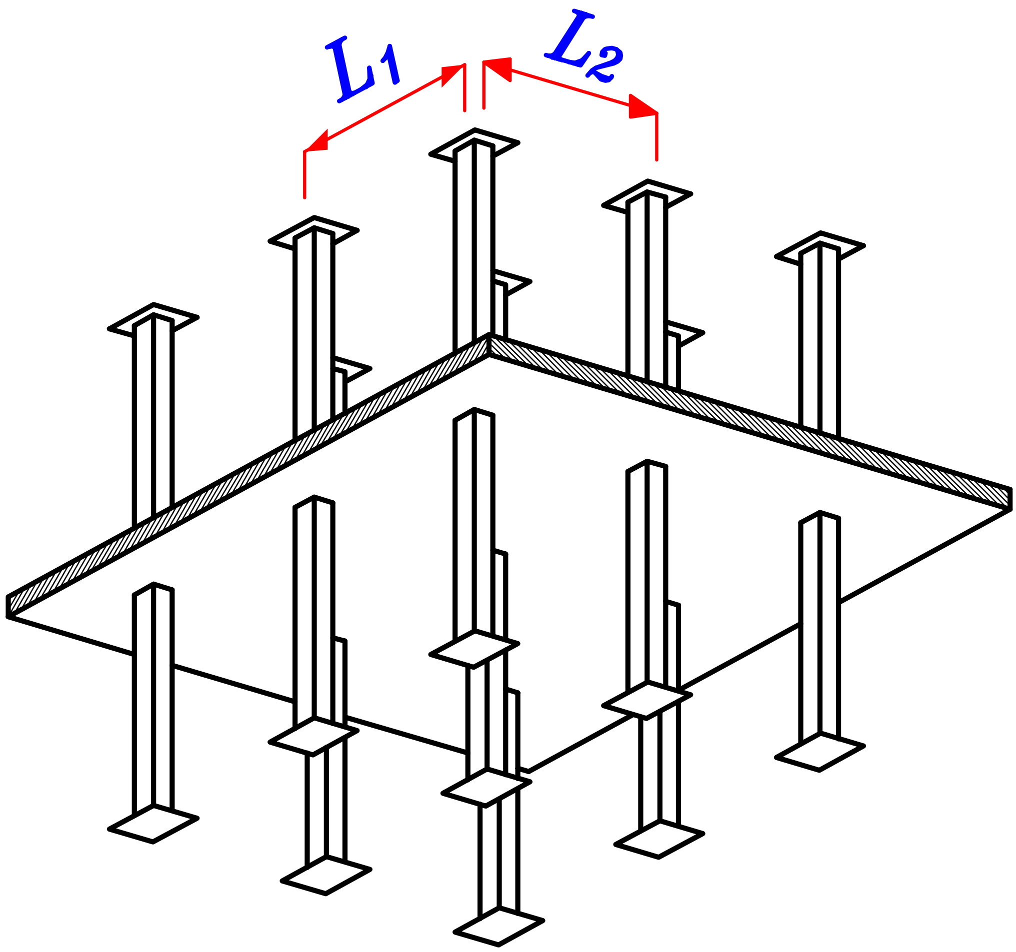
① نعتبر أن البلاطه و الاعمده تعمل كأنها Space Frame

أى أن العزوم تنتقل من البلاطه الى الاعمده و العكس .



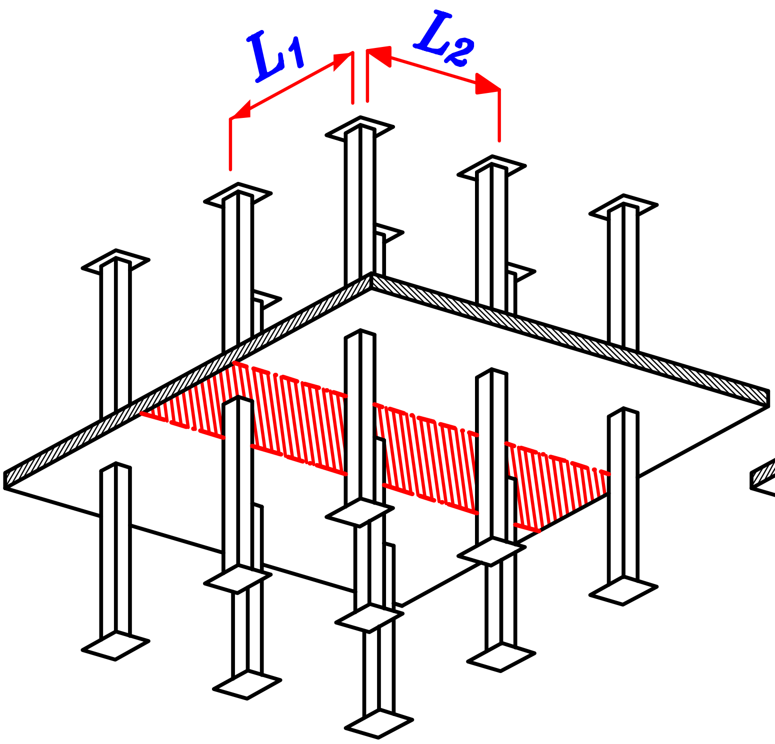
Space Frame

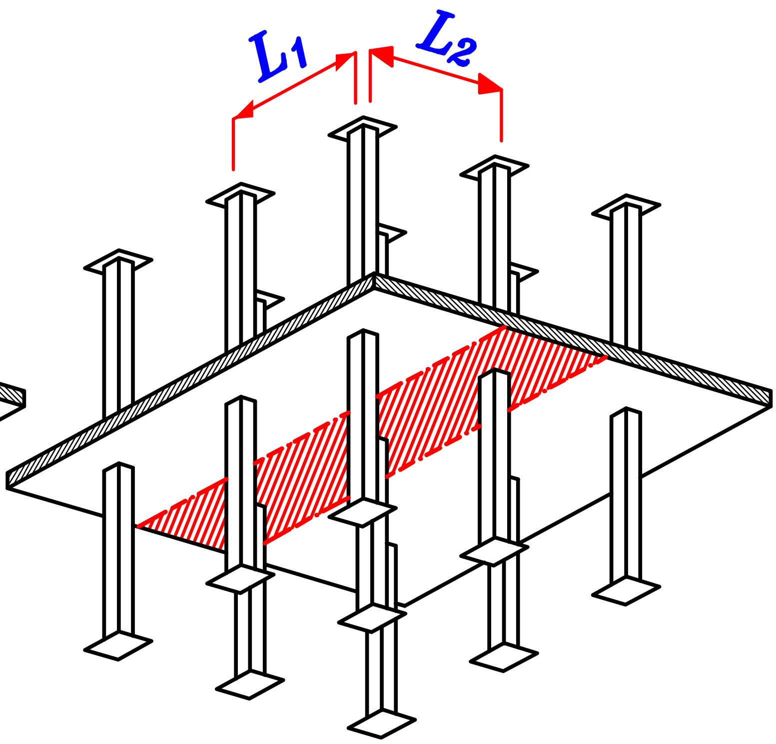
② نفترض أن ال *Frame* يتكون من دور واحد و ذلك لتسهيل الحل .



③ يتم تحويل ال *Space Frame* إلى *Plane Frame*

بأخذ شريحه بعرض الباكيه $(L_1 \text{ or } L_2)$ فى الإتجاهين .





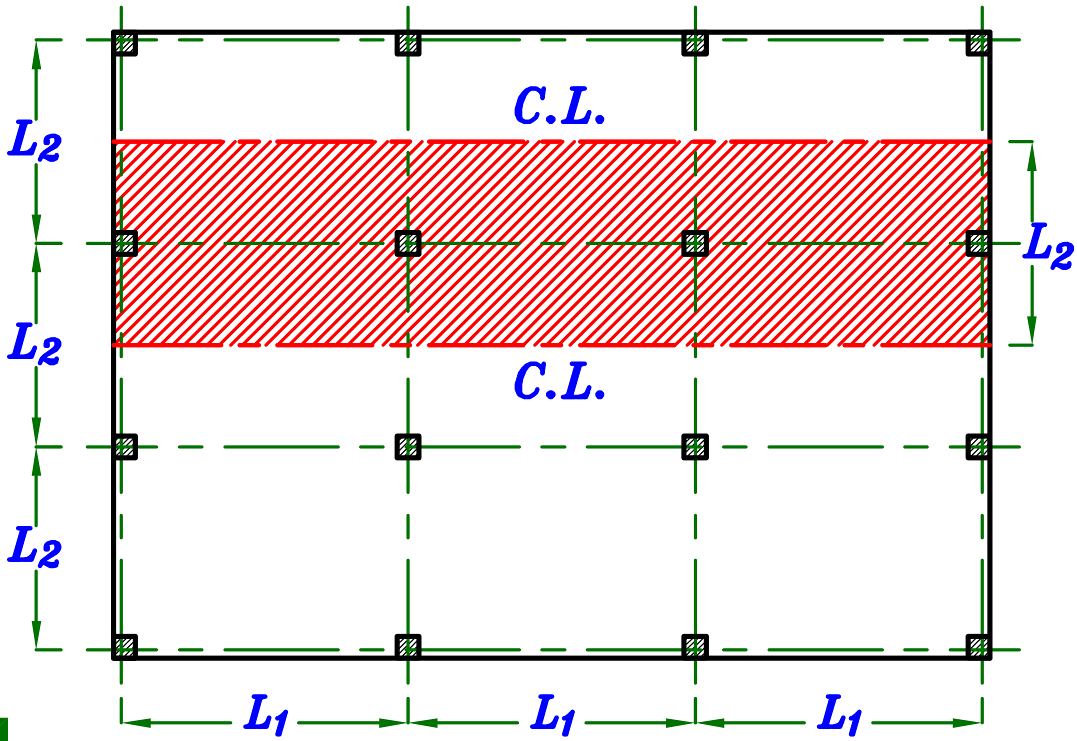
④ رسم ال *Plane Frames* كل على حده .

Plane Frame at Long Direction.

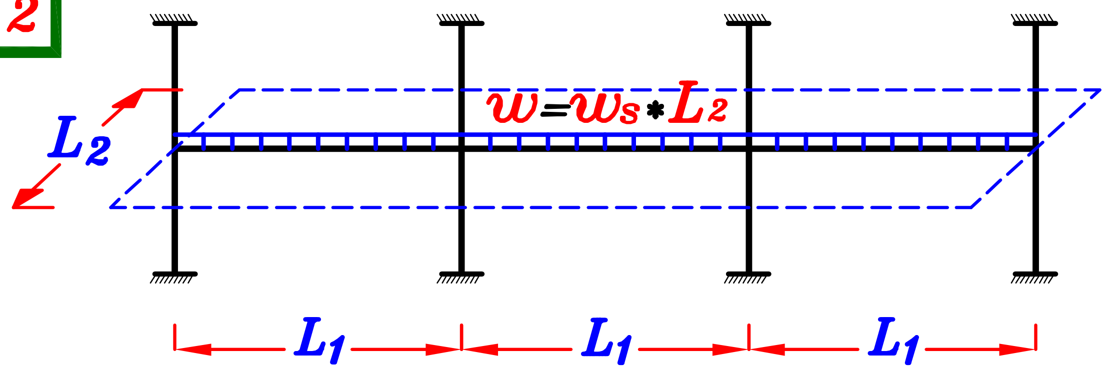
Without Drop Panel

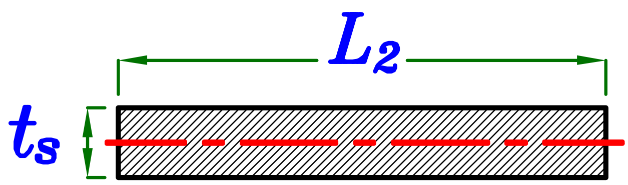
$Span = L_1$

$Frame\ Width = L_2$

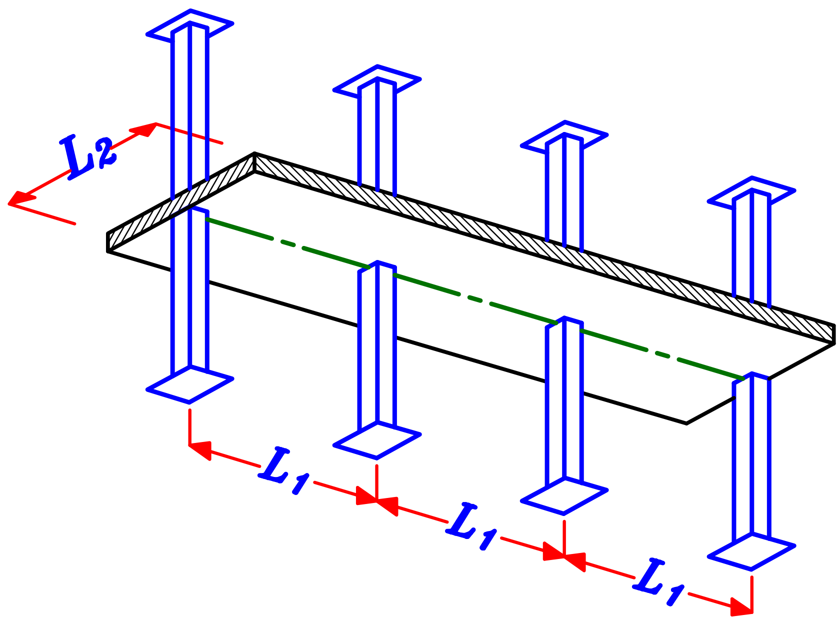


$$\boxed{w = w_s * L_2}$$



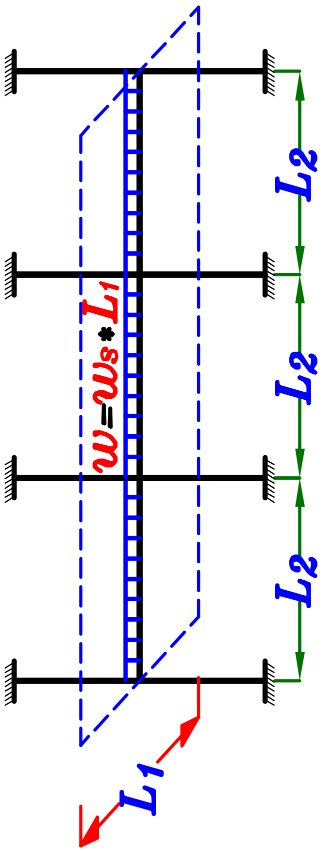


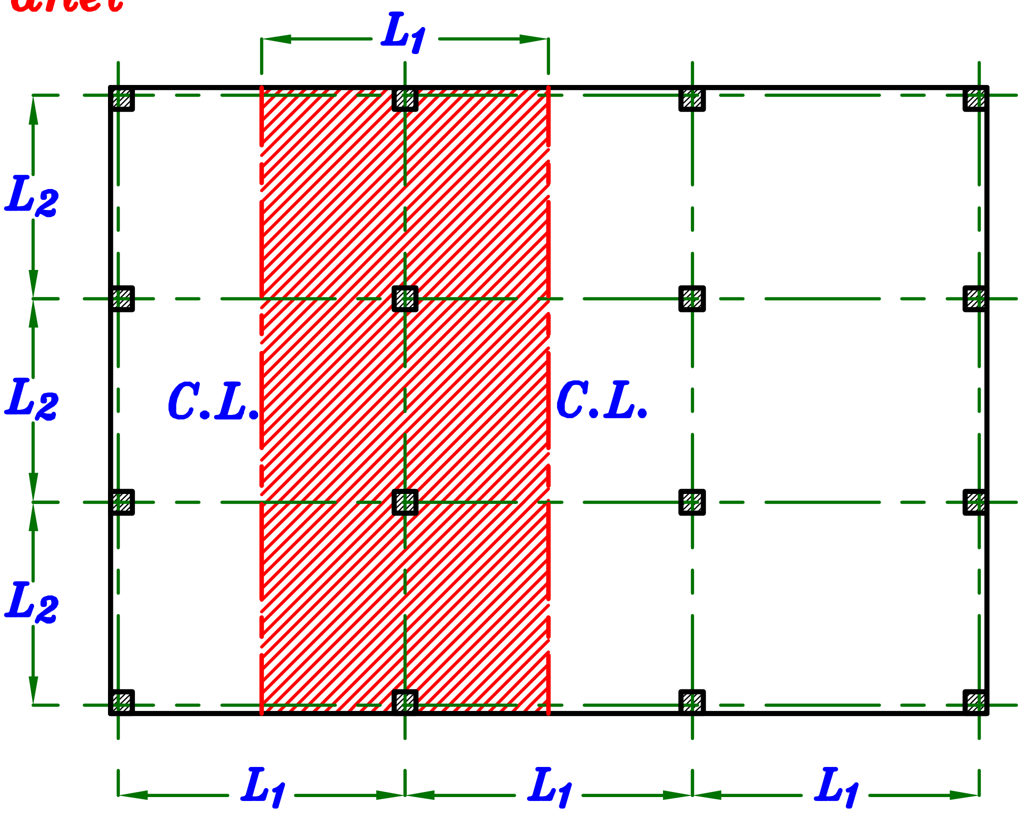
$$\boxed{I_s = \frac{L_2 * t_s^3}{12}}$$



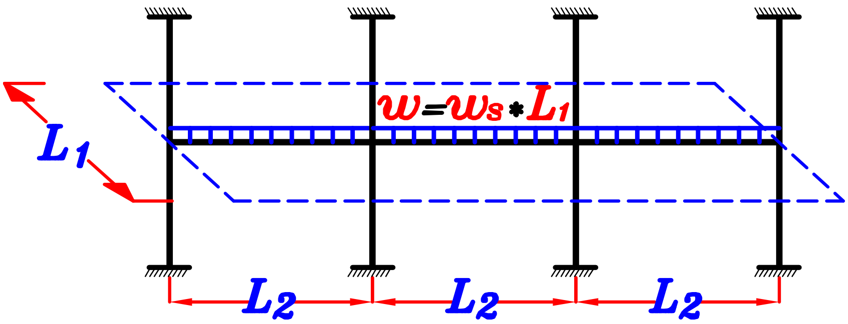
Plane Frame at Short Direction.

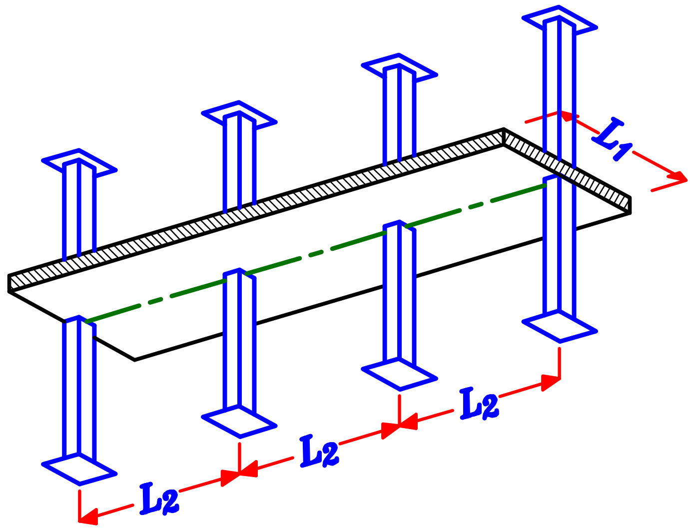
Without Drop Panel



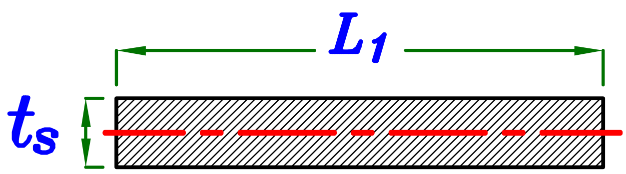


Span $= L_2$ Frame Width $= L_1$ $\boxed{w = w_s * L_1}$





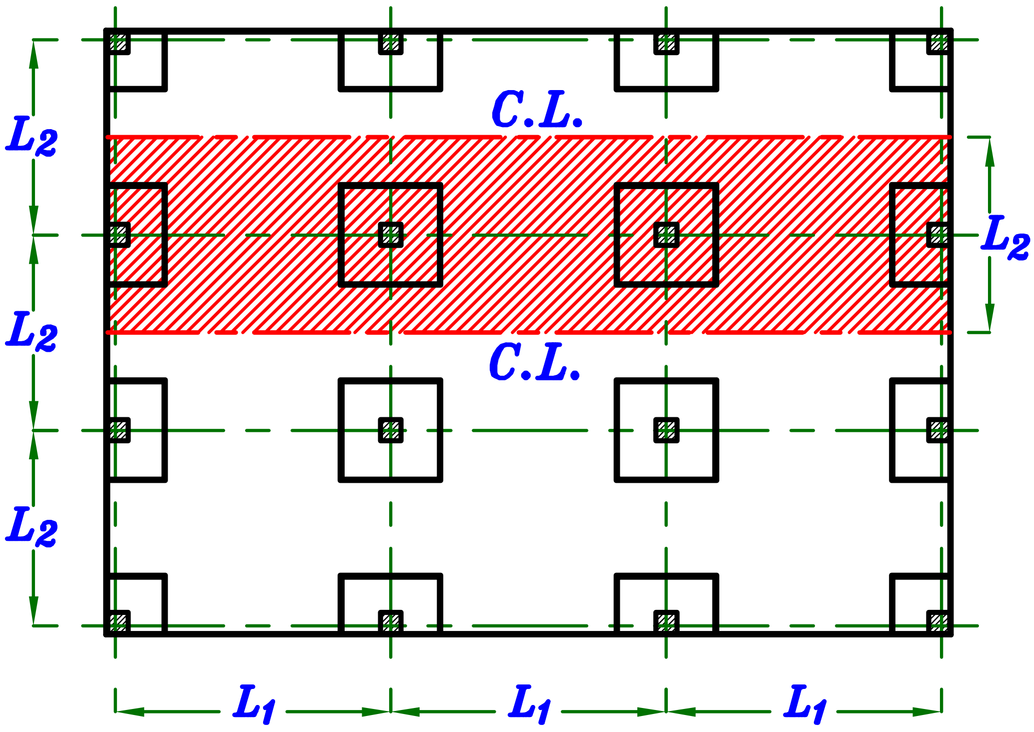
$$\boxed{I_s = \frac{L_1 * t_s^3}{12}}$$



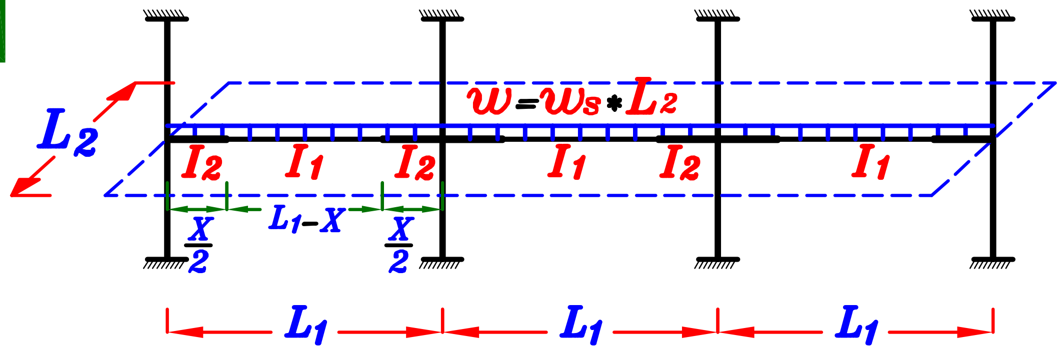
Plane Frame at Long Direction. With Drop Panel

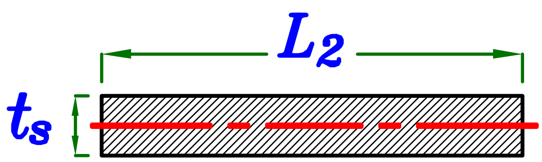
Span $= L_1$

Frame Width $= L_2$

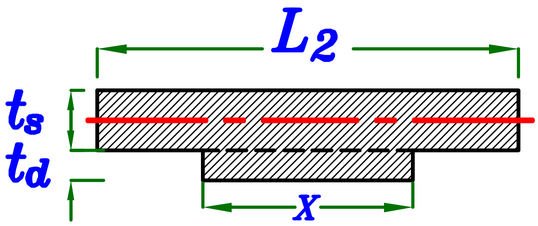


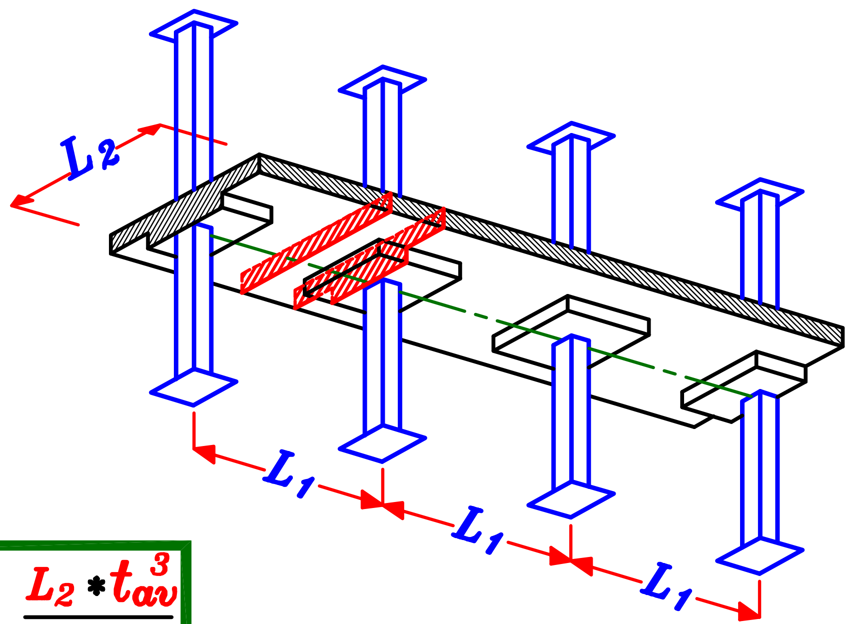
$$w = w_s * L_2$$





$$I_1 = \frac{L_2 * t_s^{\,3}}{12}$$



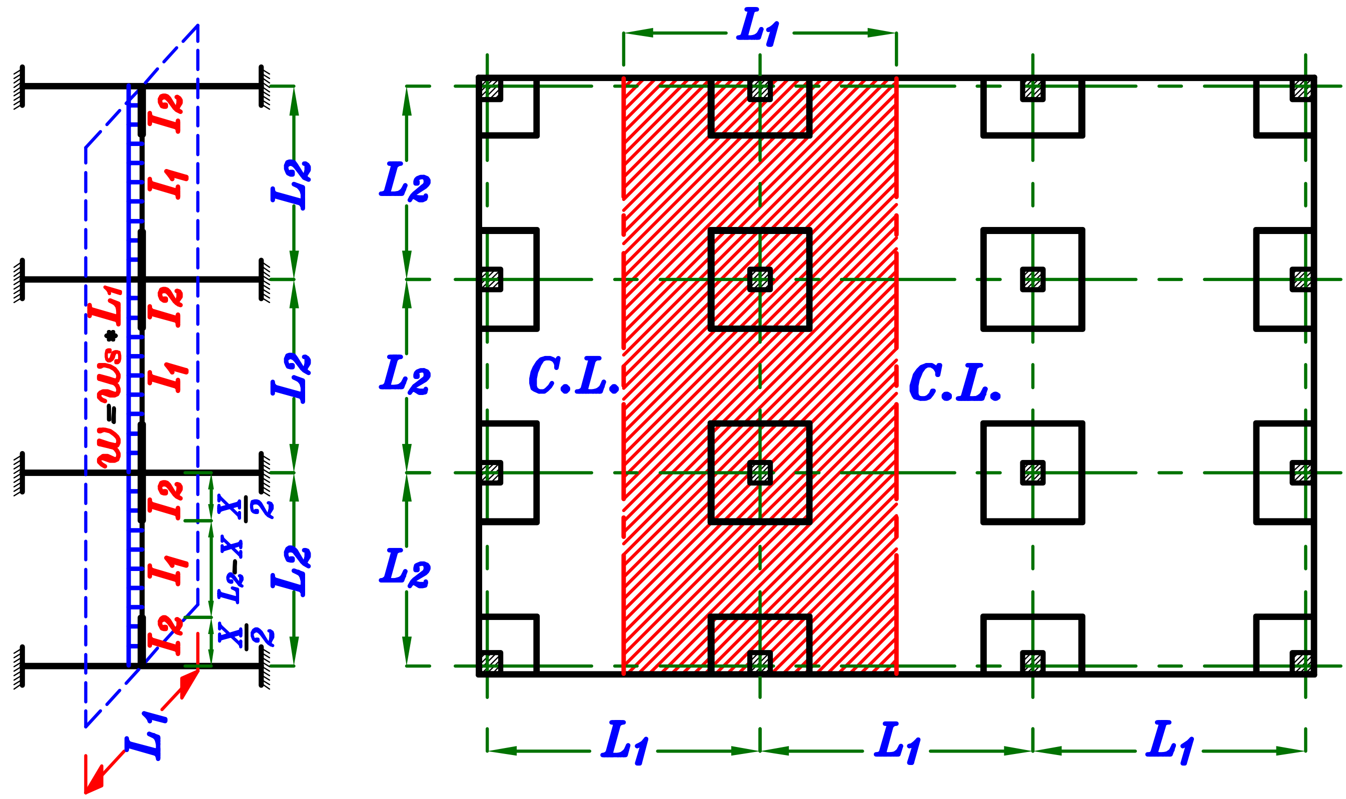


$$t_{av} = t_s + \frac{t_d}{4}$$

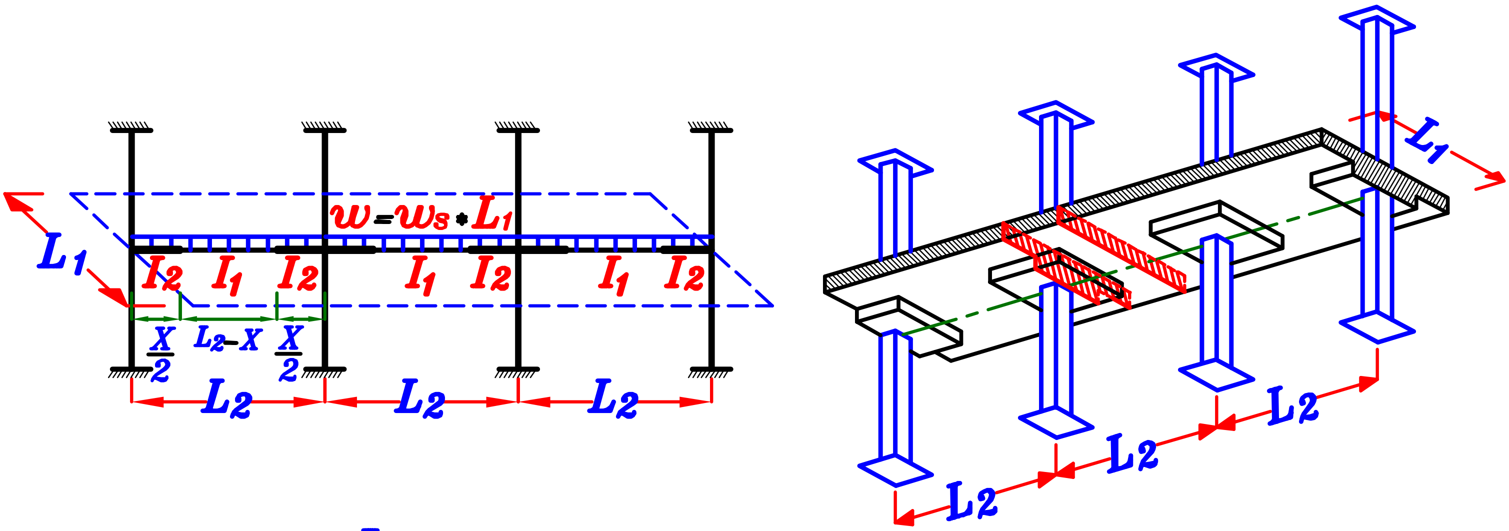
$$I_2 = \frac{L_2 * t_{av}^{\,3}}{12}$$

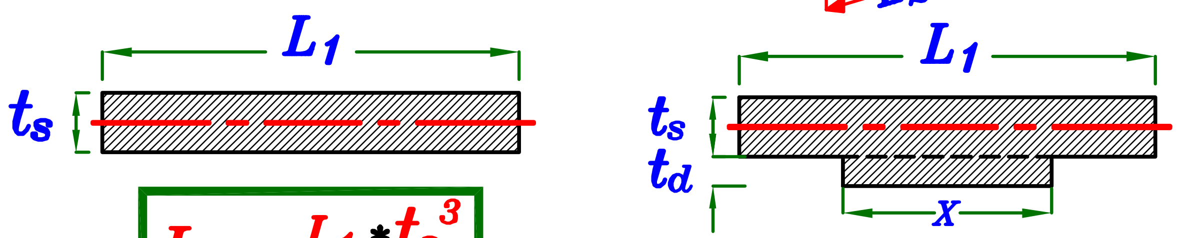
$$I_{s\,av} = \frac{I_1 (L_1 - X) + I_2 (X)}{L_1}$$

Plane Frame at Short Direction. With Drop Panel



Span $= L_2$ Frame Width $= L_1$ $\boxed{w = w_s * L_1}$





$$\boxed{I_1 = \frac{L_1 * t_s^3}{12}}$$

$$\boxed{t_{av} = t_s + \frac{t_d}{4}}$$

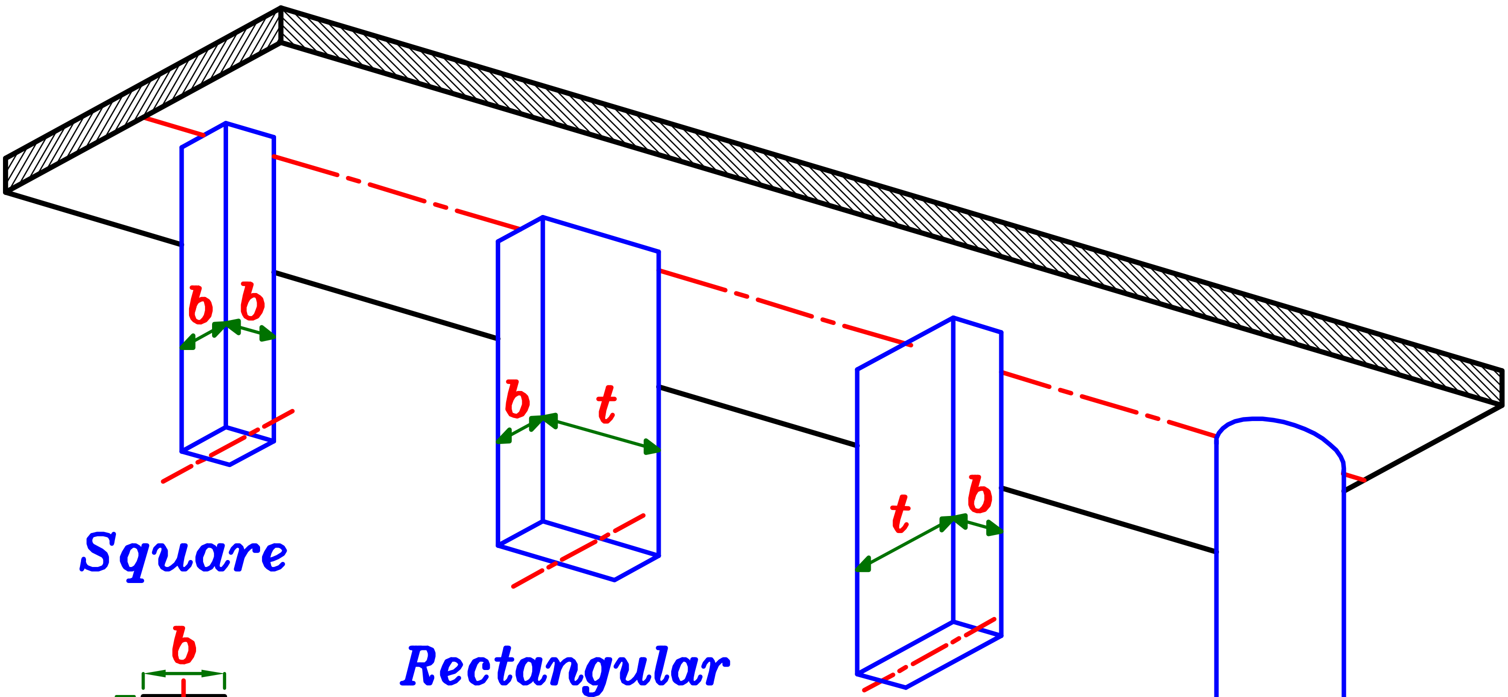
$$\boxed{I_2 = \frac{L_1 * t_{av}^3}{12}}$$

$$\boxed{I_{s\,av} = \frac{I_1 (L_2 - X) + I_2 (X)}{L_2}}$$

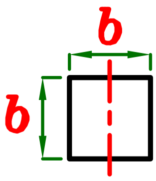
Moment Distribution Method.

① Calculate Moment of Inertia For Slabs & Columns.

ⓐ Columns.

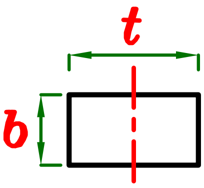


Square



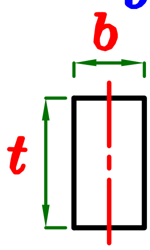
$$I = \frac{(b)^4}{12}$$

Rectangular



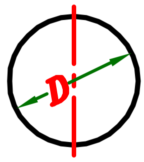
$$I = \frac{b(t)^3}{12}$$

Rectangular



$$I = \frac{t(b)^3}{12}$$

Circular



$$I = \frac{\pi(D)^4}{64}$$

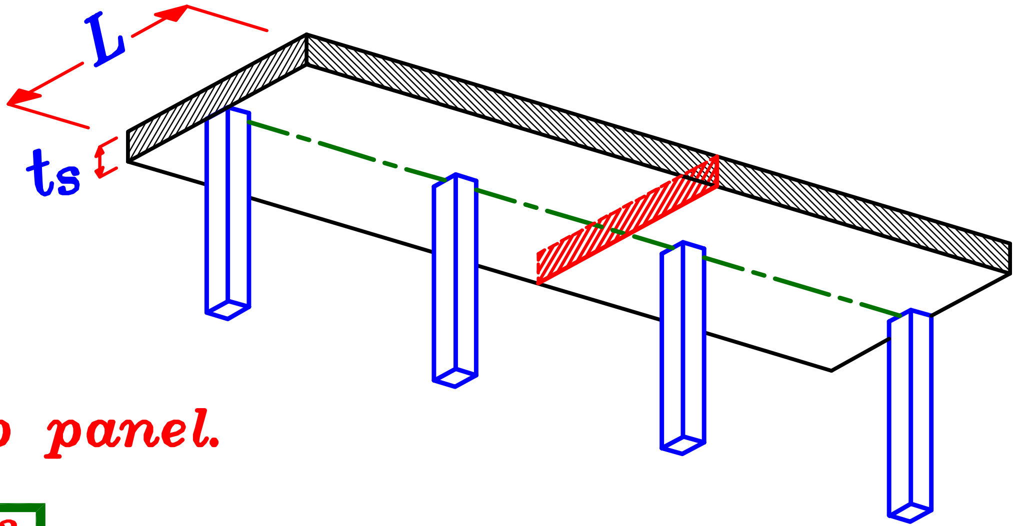
$$\boxed{I_c = \Psi * I}$$

يتم ضرب قيمه I للعمود فى معامل تصحيح يساوى Ψ

$\Psi = 0.6 * \left(\frac{L_2}{L_1}\right)^2$ للعمود الطرفى $\xrightarrow{\text{عاده تؤخذ}}$ $\boxed{\Psi = 0.6}$

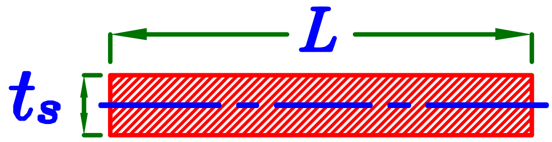
$\Psi = 0.3 * \left(\frac{L_2}{L_1}\right)^2$ للعمود الوسطى $\xrightarrow{\text{عاده تؤخذ}}$ $\boxed{\Psi = 0.3}$

ⓑ Slabs.

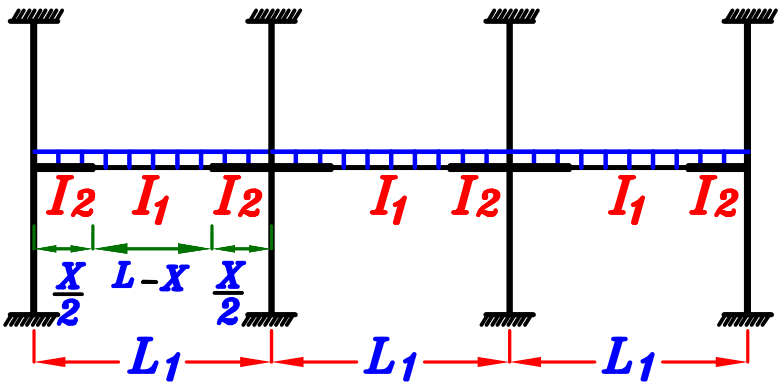


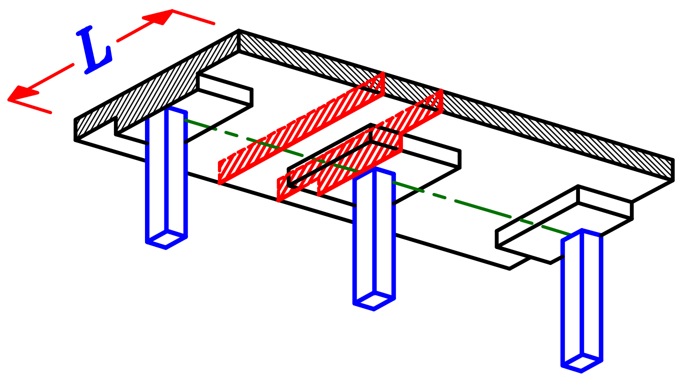
1 Without drop panel.

$$I_s = \frac{L * t_s^3}{12}$$



2 With drop panel.

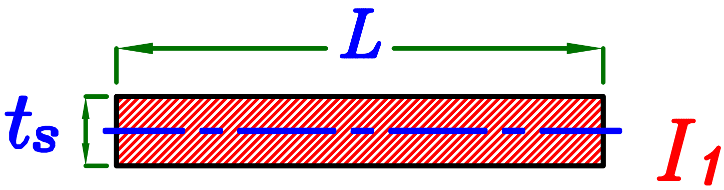




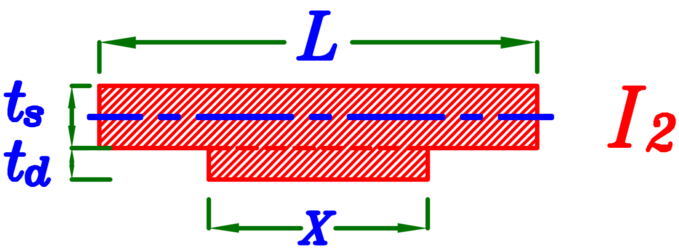
$$I_1 = \frac{L * t_s^3}{12}$$

$$I_2 = \frac{L * t_{av}^3}{12}$$

where: $$t_{av} = t_s + \frac{t_d}{4}$$



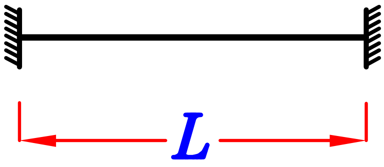
$$I_s = \frac{I_1 (L_1 - X) + I_2 (X)}{L_1}$$



② Calculate the stiffnesss For each member.

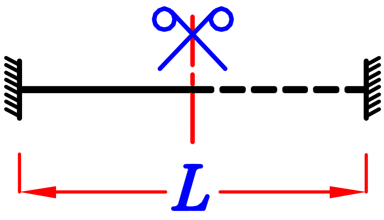
For Slabs.

$$K_s = \frac{I_s}{L}$$



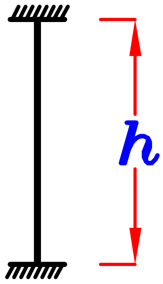
$$K_s = \frac{1}{2} * \frac{I_s}{L}$$

IF Symmetric



For Columns.

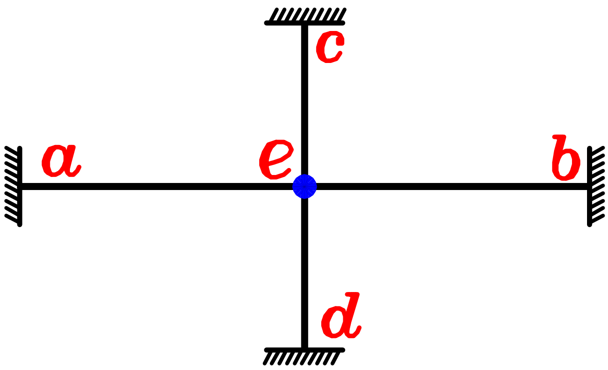
$$K_c = \frac{I_c}{h}$$



③ Calculate the Distribution Factors. (D.F.)

For Joint (e)

$$\Sigma K = K_{ea} + K_{eb} + K_{ec} + K_{ed}$$



$$D.F.(ea) = \frac{K_{ea}}{\Sigma K} \quad , \quad D.F.(eb) = \frac{K_{eb}}{\Sigma K}$$

$$D.F.(ec) = \frac{K_{ec}}{\Sigma K} \quad , \quad D.F.(ed) = \frac{K_{ed}}{\Sigma K}$$

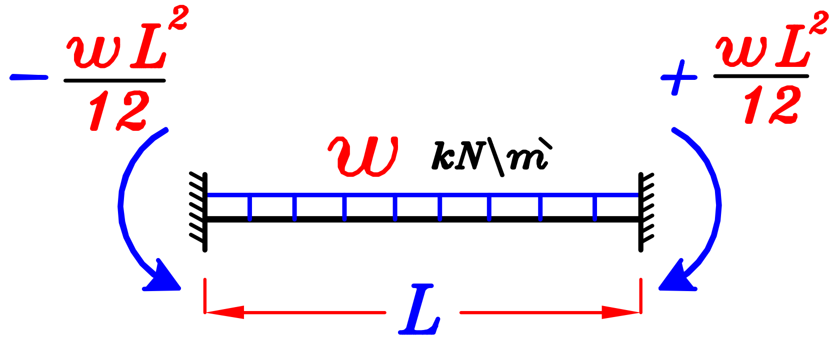
Note: For any Joint $\Sigma\, D.F. = 1.0$

$$D.F.(\text{Cantilever}) = \text{Zero}$$

$$D.F.(\text{Fixation}) = \text{Zero}$$

④ Calculate Fixed End Moment For the Slab.

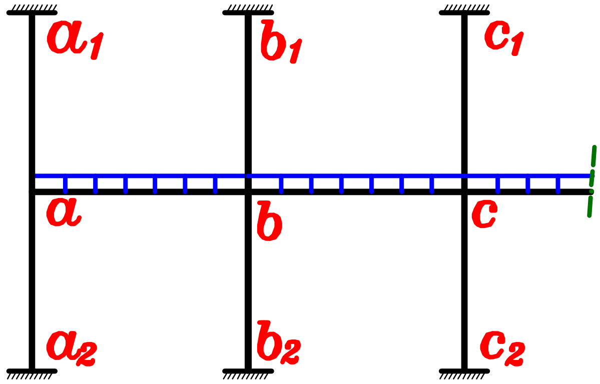
$$\boxed{F.E.M. = \frac{wL^2}{12}}$$



مع مراعاه الاشارات

عكس عقارب الساعه　　　　مع عقارب الساعه

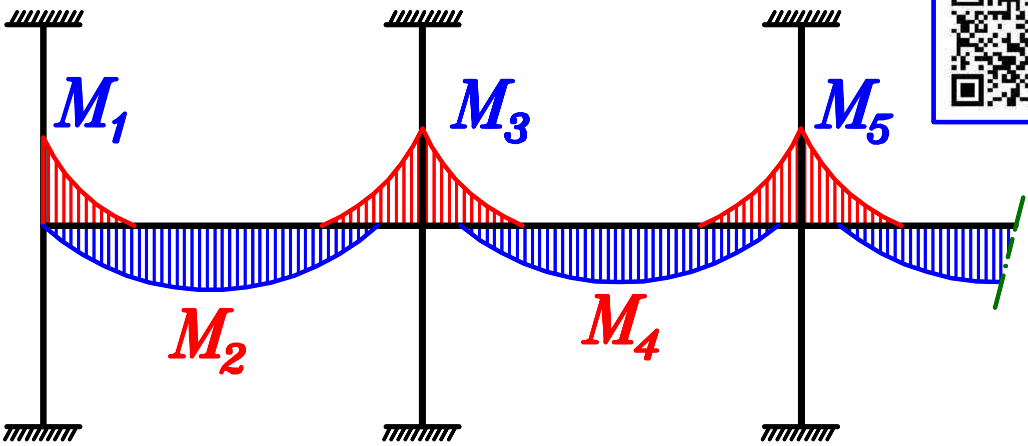
⑤ Make the Table of Moment Distribution.



Joint	a_1	a			a_2	b_1	b				b_2	c_1	
member	$a_1\text{-}a$	$a\text{-}a_1$	$a\text{-}b$	$a\text{-}a_2$	$a_2\text{-}a$	$b_1\text{-}b$	$b\text{-}b_1$	$b\text{-}a$	$b\text{-}c$	$b\text{-}b_2$	$b_2\text{-}b$	$c_1\text{-}c$	$c\text{-}$
D.F.	—	✓	✓	✓	—	—	✓	✓	✓	✓	—	—	✓
F.E.M.	—	—	−✓	—	—	—	—	+✓	−✓	—	—	—	—
B.M.	—	✓	✓	✓	—	—	✓	✓	✓	✓	—	—	✓
C.O.M.	$\frac{1}{2}M_{a\text{-}a_1}$	—	$\frac{1}{2}M_{b\text{-}a}$	—	$\frac{1}{2}M_{a\text{-}a_2}$	$\frac{1}{2}M_{b\text{-}b_1}$	—	$\frac{1}{2}M_{a\text{-}b}$	$\frac{1}{2}M_{c\text{-}b}$	—	$\frac{1}{2}M_{b\text{-}b_2}$	$\frac{1}{2}M_{c\text{-}c_1}$	—
B.M.	—	✓	✓	✓	—	—	✓	✓	✓	✓	—	—	✓
C.O.M.	$\frac{1}{2}M_{a\text{-}a_1}$	—	$\frac{1}{2}M_{b\text{-}a}$	—	$\frac{1}{2}M_{a\text{-}a_2}$	$\frac{1}{2}M_{b\text{-}b_1}$	—	$\frac{1}{2}M_{a\text{-}b}$	$\frac{1}{2}M_{c\text{-}b}$	—	$\frac{1}{2}M_{b\text{-}b_2}$	$\frac{1}{2}M_{c\text{-}c_1}$	—
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮
M_F	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓

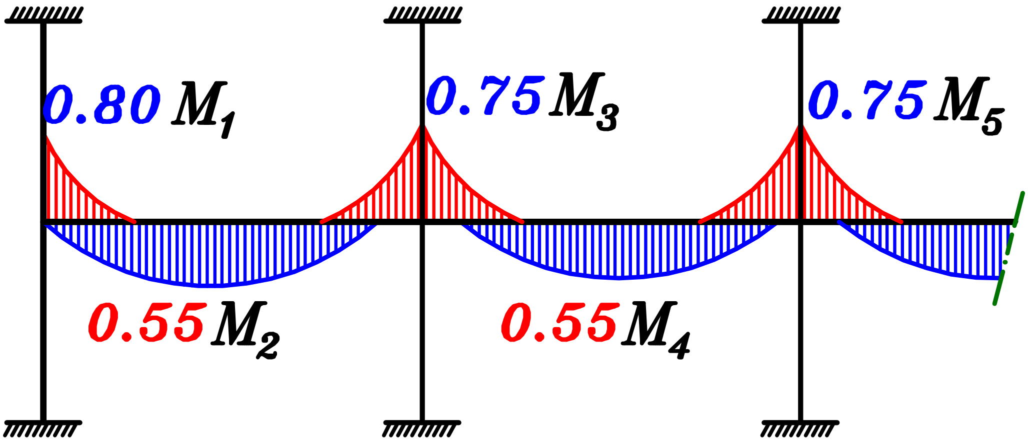
⑤ يتم حل ال *Plane Frames* بطريقه *Moment Distribution*

M_{Total}

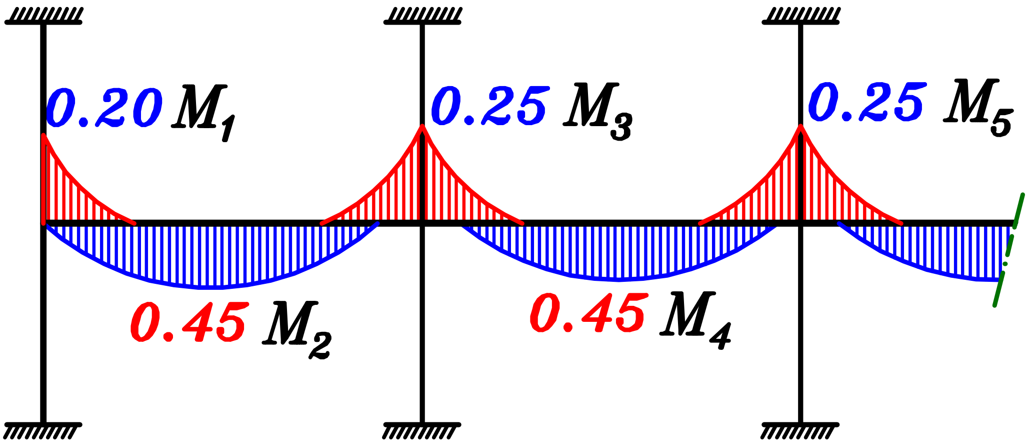


⑥ توزيع M_{Total} على ال *Column Strip* و ال *Field Strip* بالنسب التاليه .

Column Strip



Field Strip

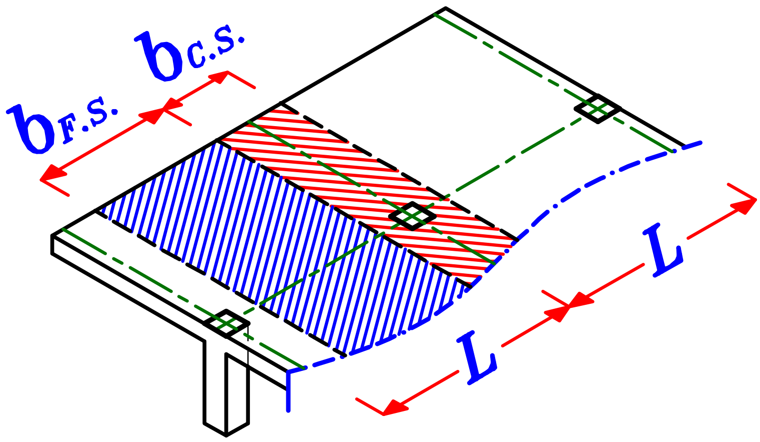


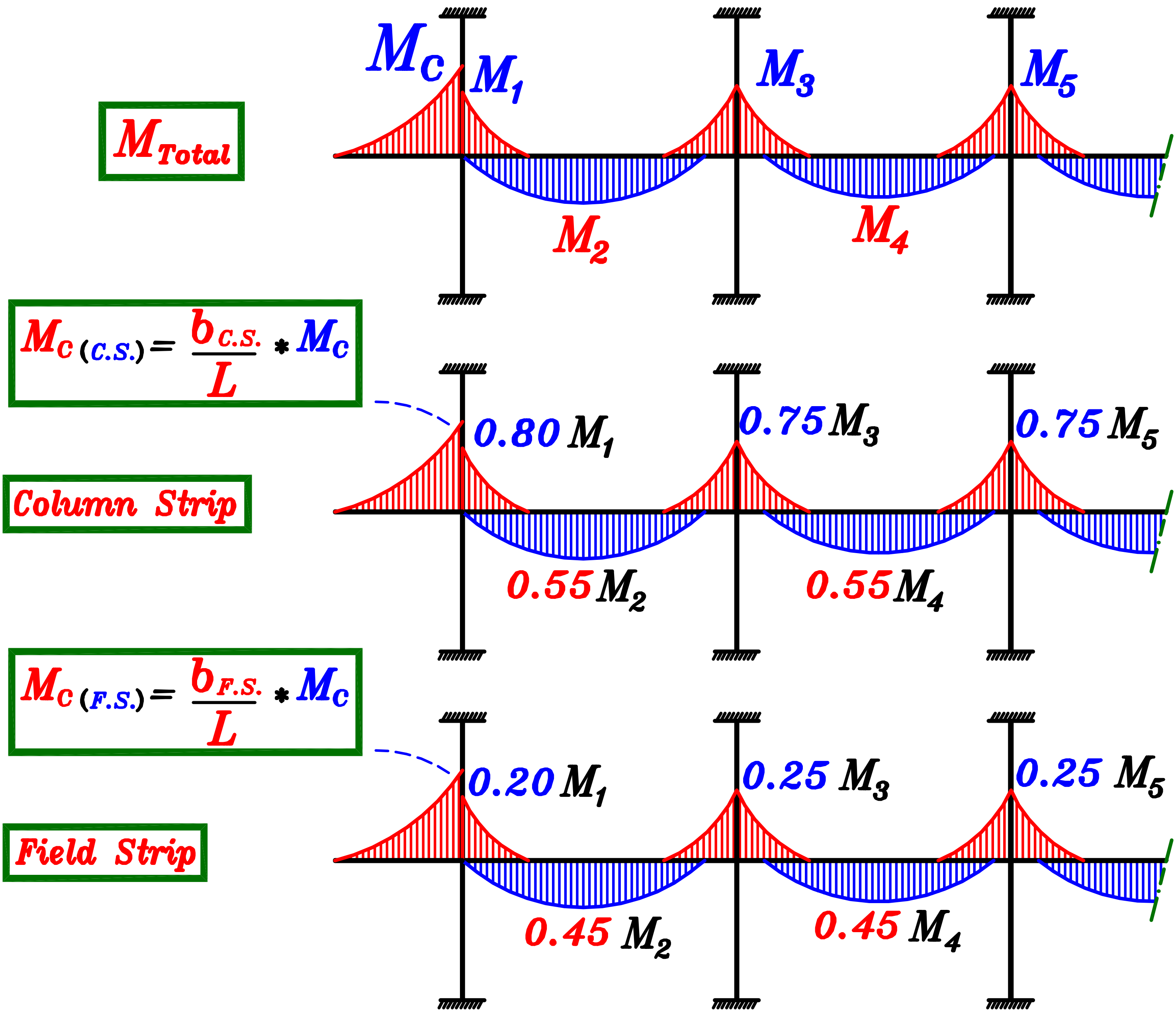
فى حاله عدم تساوى عرض *Column Strip* مع ال *Field Strip*
يتم عمل ال *Modification* كما سبق .

في حاله وجود Cantilever مع ال Frame
بعد حساب العزم M_c للـ Cantilever
الذي هو على الباكيه كلها يتم توزيعه على ال Column Strip و ال Field Strip
بنسبه عرض كل شريحه الى الشريحه الكليه .

$$\boxed{M_{c\,(C.S.)} = \frac{b_{C.S.}}{L} * M_c}$$

$$\boxed{M_{c\,(F.S.)} = \frac{b_{F.S.}}{L} * M_c}$$





Special Cases.

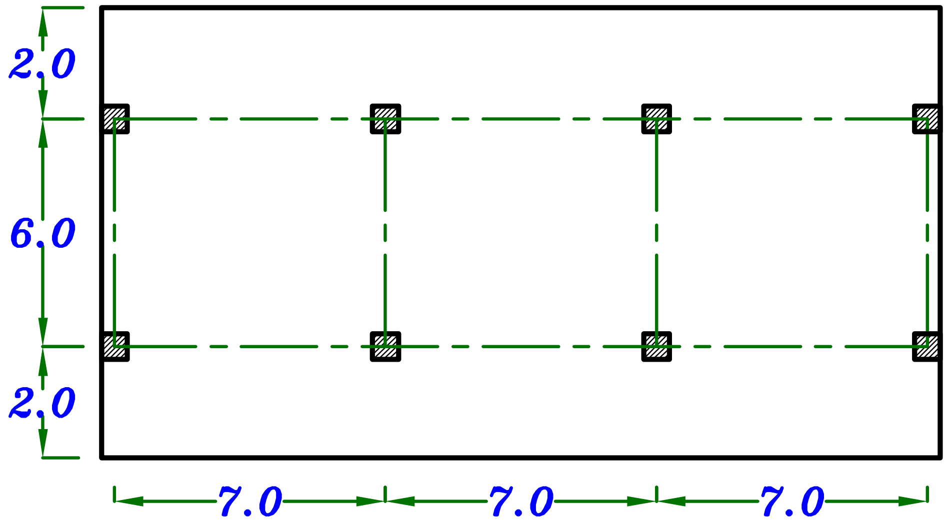
فى حاله وجود *Cantilever* مع باكيه واحده فقط.
يتم اعتبار عرض الـ *Cantilever* أنه من شريحه الـ *Column Strip*

Example.

$L_1 = 7.0\ m$

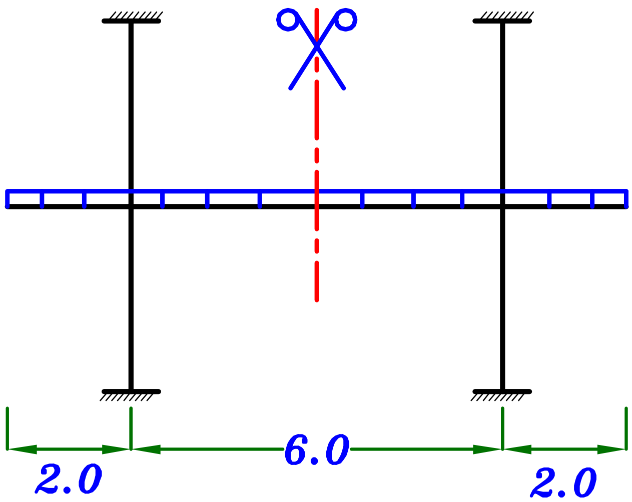
$L_2 = 6.0\ m$

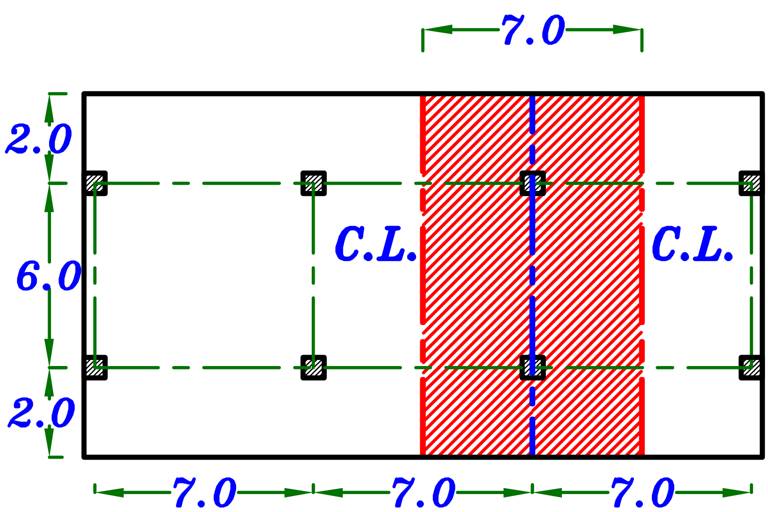
$L_c = 2.0\ m$



Strip in short direction.

بعد اختيار *C.L.* فى المنتصف يتم أخذ شريحه التصميم الكليه من *C.L.* البلاطه الى *C.L.* البلاطه



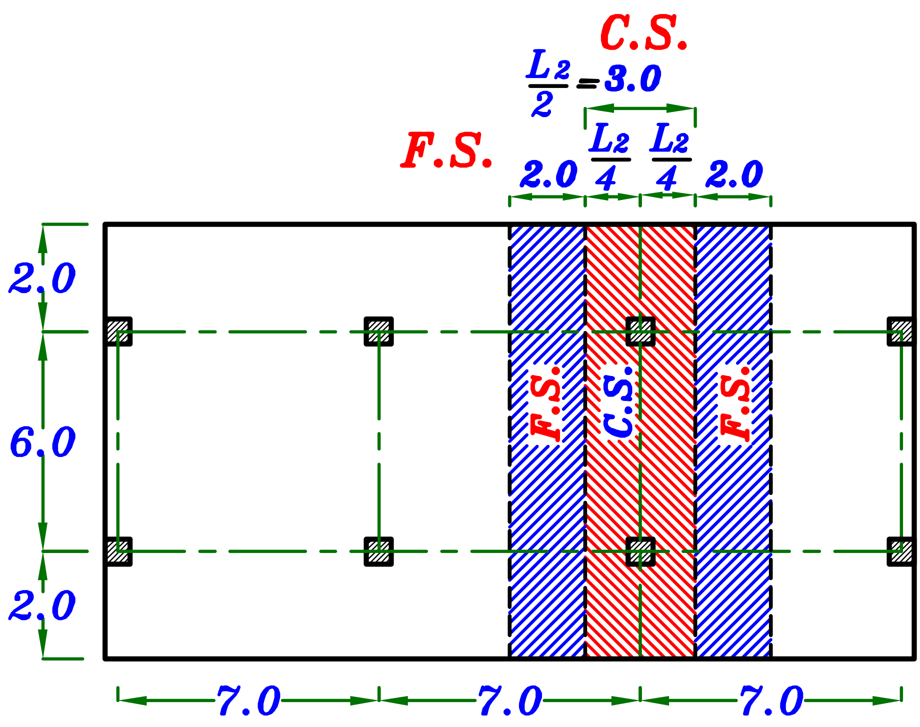


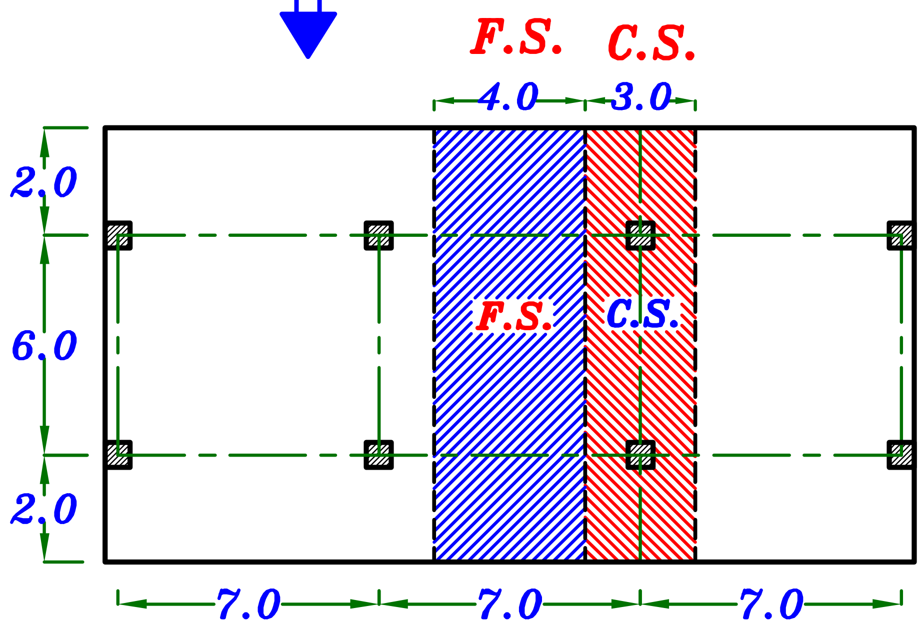
يتم حل الشريحه بطريقه *Frame analysis*
لان عدد البواكى أقل من ٣ بواكى .

يتم توزيع العزم على كلا من ال Column Strip و ال Field Strip

$$b_{C.S.} = \frac{L_2}{4} + \frac{L_2}{4} = \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

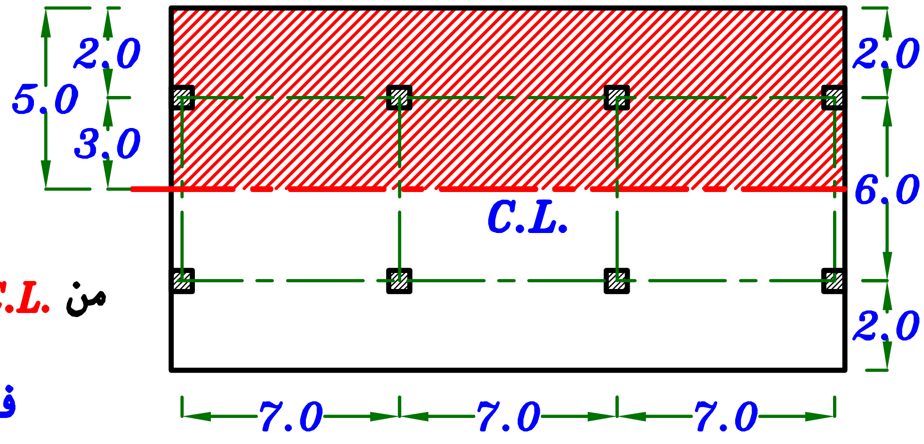
$$b_{F.S.} = \text{Total Strip width} - b_{C.S.} = 7.0 - 3.0 = 4.0\ m$$





$$M.F. = \frac{\text{Field Strip العرض الحقيقى لل}}{\text{C.L. to C.L. نصف عرض الشريحه من}} = \frac{4.0}{\frac{1}{2} * 7.0} = 1.14$$

Strip in Long direction.



يتم أخذ شريحه التصميم الكليه

من $C.L.$ البلاطه الى نهايه الـ Cantilever

فيكون عرض شريحه التصميم الكليه

$$Total\ Strip\ width = \frac{6.0}{2.0} + 2.0 = 5.0\ m$$

يؤخذ عرض الـ Column strip

$$\boxed{b_{C.S.} = \frac{L_2}{4} + Width\ of\ the\ Cantilever}$$

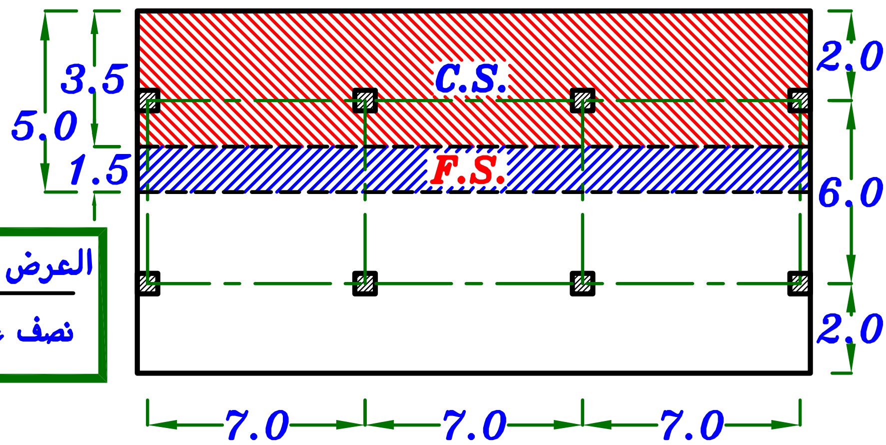
$$b_{C.S.} = \frac{6.0}{4} + 2.0 = 3.50\ m$$

و يؤخذ عرض الـ Field strip

$$\boxed{b_{F.S.} = Total\ Strip\ width - b_{C.S.}}$$

$$b_{F.S.} = 5.0 - 3.50 = 1.50\ m$$

Modification Factor For Field Strip



$$\boxed{M.F. = \frac{\text{العرض الحقيقى للـ Field Strip}}{\text{نصف عرض الشريحه من C.L. to C.L.}}}$$

$$M.F. = \frac{1.5}{\frac{1}{2} * 5.0} = 0.60$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * Modification\ Factor}$$

ثم يتم اعاده حساب عزم الـ $F.S.$ بحيث يظل العزم الكلى ثابت .

$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

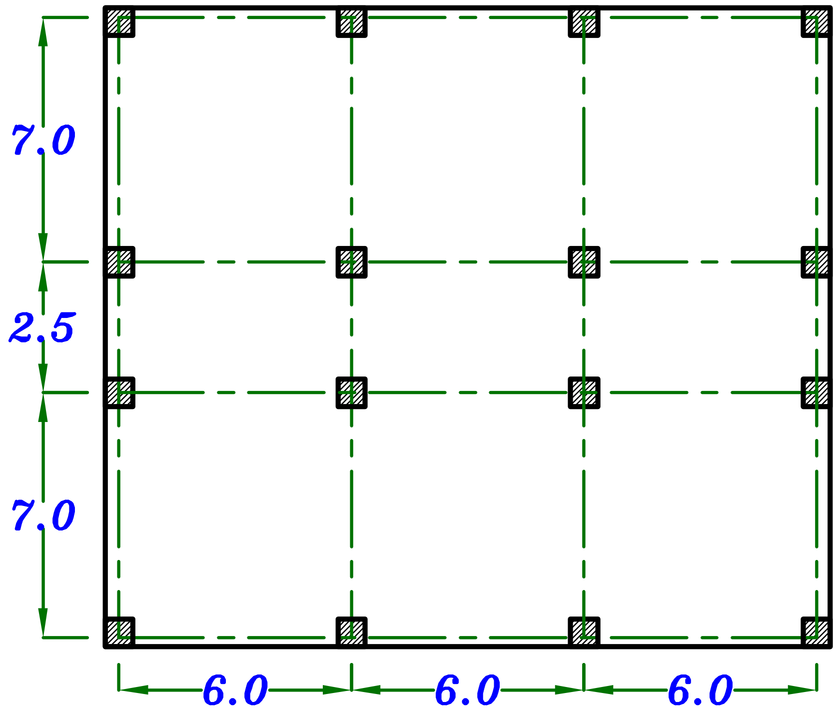
Special Cases.

فى حاله وجود باكيه صغيره بين باكيتين كبيرتين .
يتم اعتبار الباكيه الصغيره بالكامل على أنها Column Strip

Example.

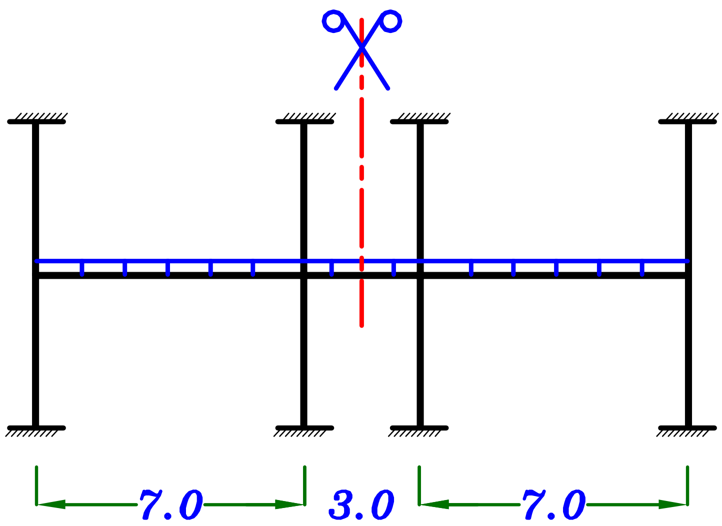
$L_1 = 7.0\ m$

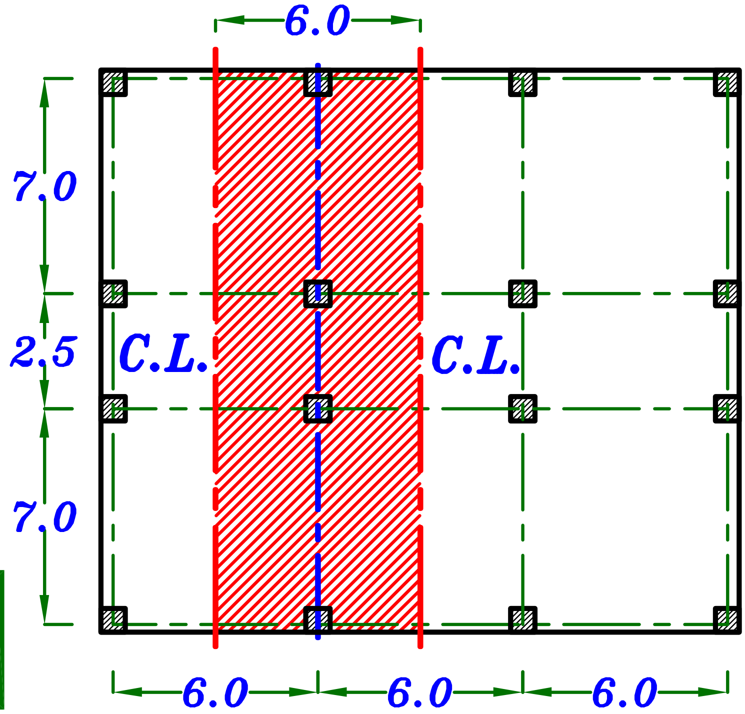
$L_2 = 6.0\ m$



Strip in Short direction.

بعد اختيار C.L. فى المنتصف يتم أخذ شرحه التصميم الكليه من C.L. البلاطه الى C.L. البلاطه



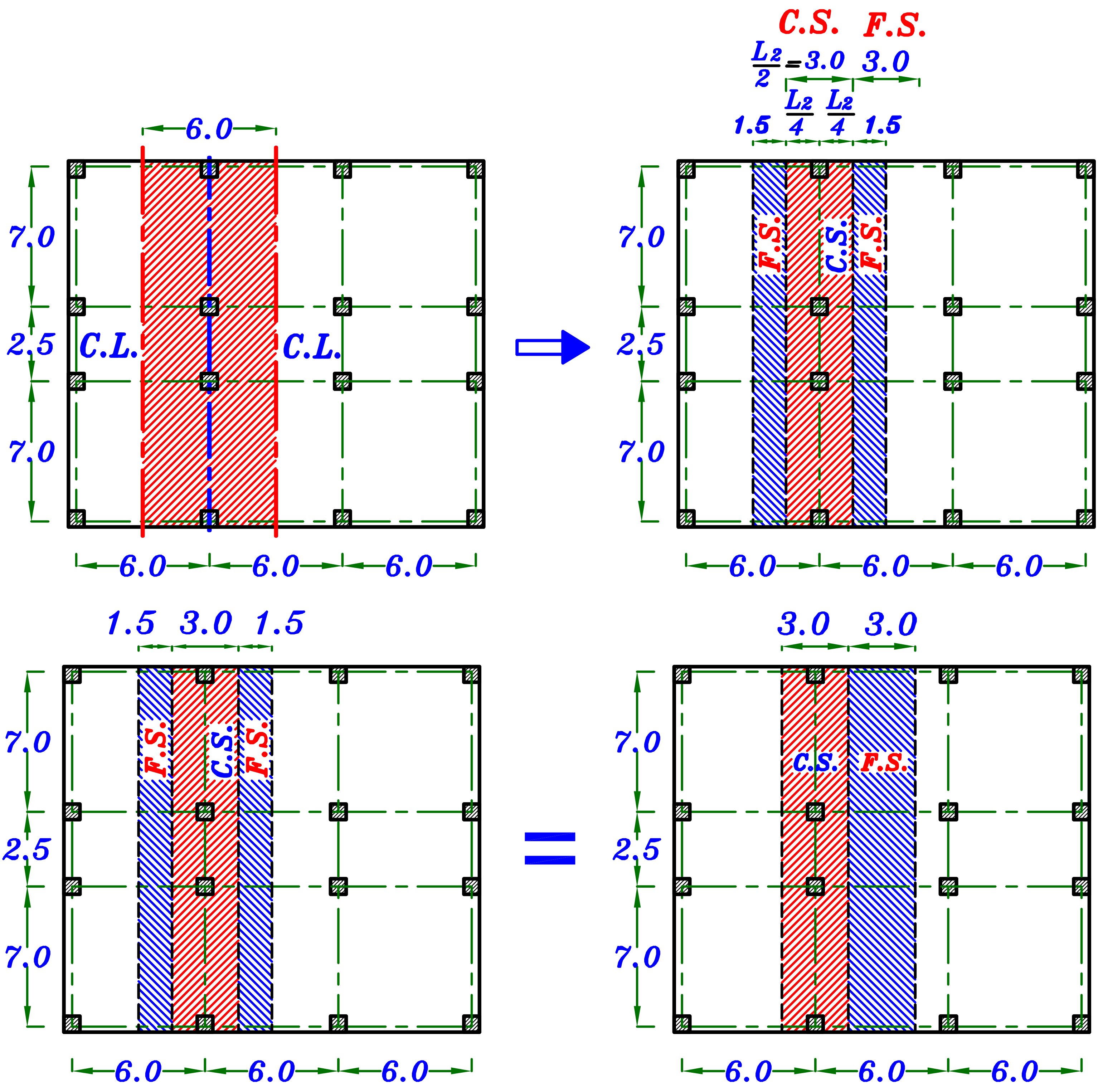


يتم حل الشريحه بطريقه Frame analysis
لان الفرق بين البواكى المجاوره أكبر من ١٠٪

يتم توزيع العزم على كلا من ال **Column Strip** و ال **Field Strip**

$$b_{C.S.} = \frac{L_2}{4} + \frac{L_2}{4} = \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

$$b_{F.S.} = \text{Total Strip width} - b_{C.S.} = 6.0 - 3.0 = 3.0\ m$$

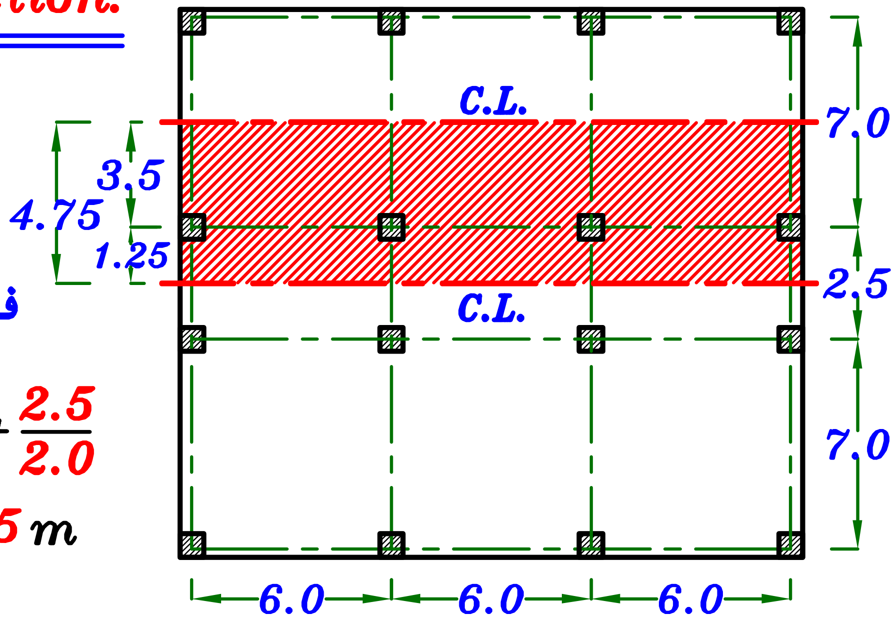


$$b_{C.S.} = b_{F.S.} \longrightarrow$$ No **Modification Factor**

Strip in Long direction.

يتم أخذ شريحه التصميم الكليه

من $C.L.$ البلاطه الى $C.L.$ البلاطه



فيكون عرض شريحه التصميم الكليه

$$\text{Total Strip width} = \frac{7.0}{2.0} + \frac{2.5}{2.0}$$

$$= 4.75\ m$$

$$\boxed{b_{C.S.} = \frac{L_2}{4} + \frac{1}{2}\ \text{width of the small span.}}$$

يؤخذ عرض ال Column strip

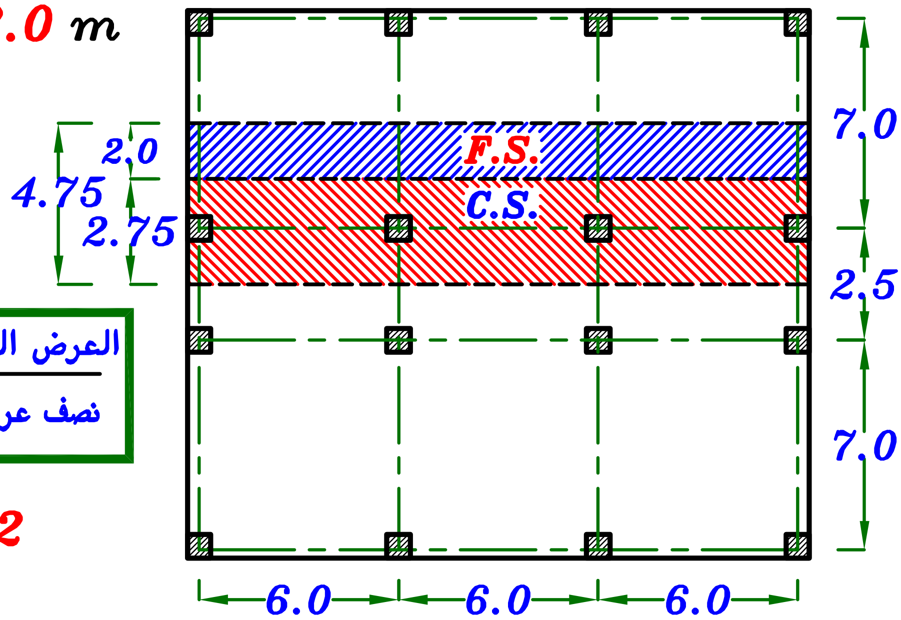
$$b_{C.S.} = \frac{6.0}{4} + \frac{2.5}{2} = 2.75\ m$$

$$\boxed{b_{F.S.} = \text{Total Strip width} - b_{C.S.}}$$

و يؤخذ عرض ال Field strip

$$b_{F.S.} = 4.75 - 2.75 = 2.0\ m$$

Modification Factor For Field Strip



$$\boxed{M.F. = \frac{\text{Field Strip العرض الحقيقى لل}}{\text{C.L. to C.L. نصف عرض الشريحه من}}}$$

$$M.F. = \frac{2.0}{\frac{1}{2} * 4.75} = 0.842$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor}}$$

ثم يتم اعاده حساب عزم ال $F.S.$ بحيث يظل العزم الكلى ثابت .

$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

Example.

The given plan shows general layout of a Flat slab Floor

The column height 4.0 m

The building consists of ground Floor and 4 typical Floors.

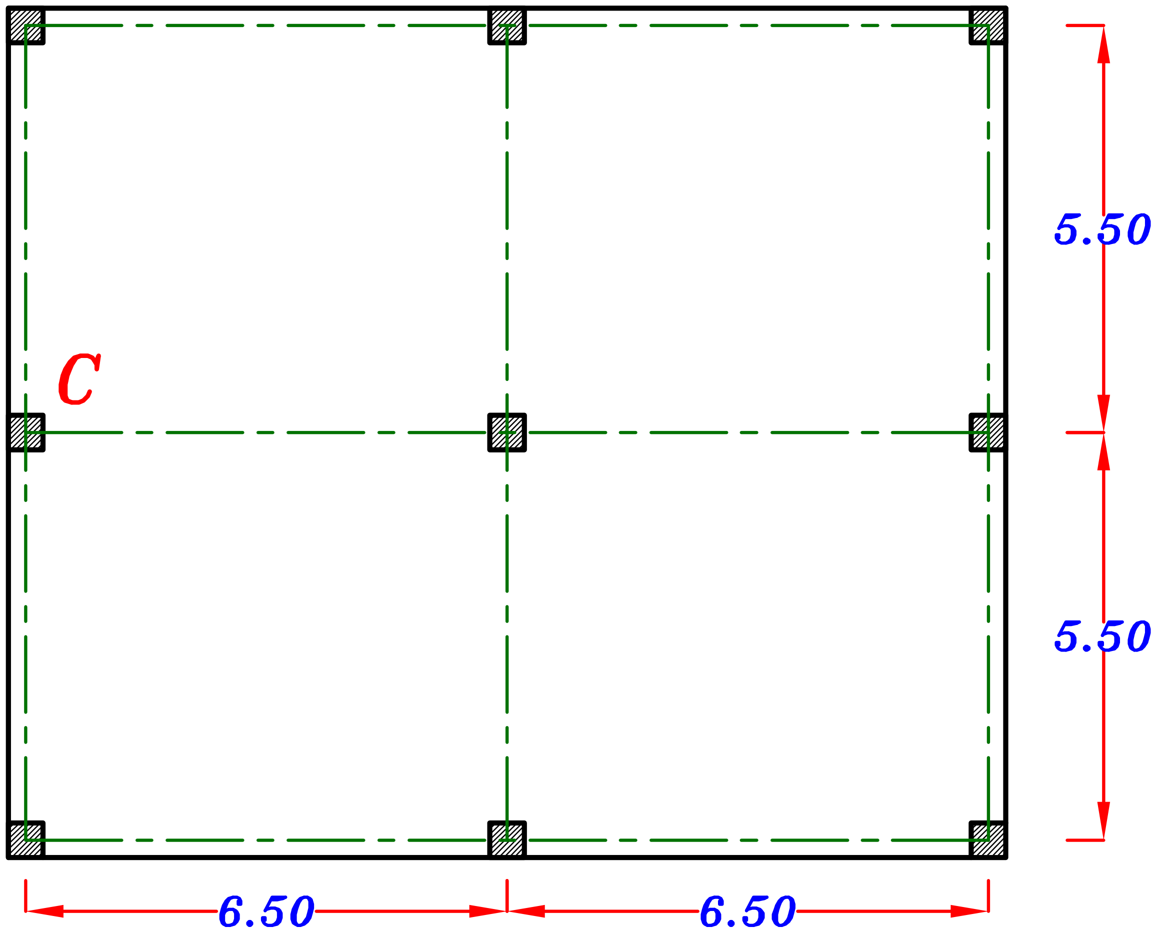
Data.

$F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

$F.C. = 1.5\ kN\backslash m^2$, $L.L. = 2.0\ kN\backslash m^2$, $Walls = 1.5\ kN\backslash m^2$

Req.

1. Using The Frame analysis method calculate the moments For both the Field strip and the column strip in both directions.
2. Design the sections of the slab. and draw details of reinforcement in plan.
3. Design the column C at ground Floor.



Solution.

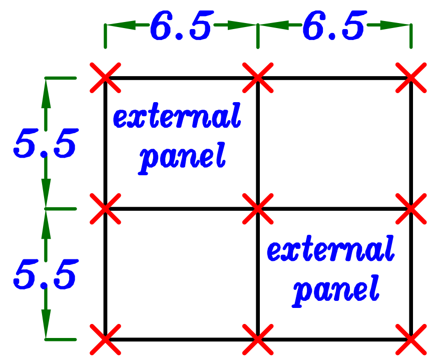
1- Concrete Dimensions.

Column dimensions.

$$b_{Col.} = \begin{cases} 300\,mm \\ \dfrac{H}{15} = \dfrac{4000}{15} = 266.6\ mm \\ \dfrac{L_1}{20} = \dfrac{6500}{20} = 325\ mm \end{cases}$$

$$\boxed{b_{Col.} = 350\ mm}$$

$(350 * 350)$



Slab Thikness.

$L_1 = 6.50\ m$

External panel $t_s = \dfrac{L_1}{32} = \dfrac{6500}{32} = 203.1\,mm$

$$\boxed{t_s = 220\,mm}$$

2- Loads on the Slab.

$$w_s = 1.4\,(t_s\,\gamma_c + F.C. + Walls) + 1.6\,(L.L.)$$

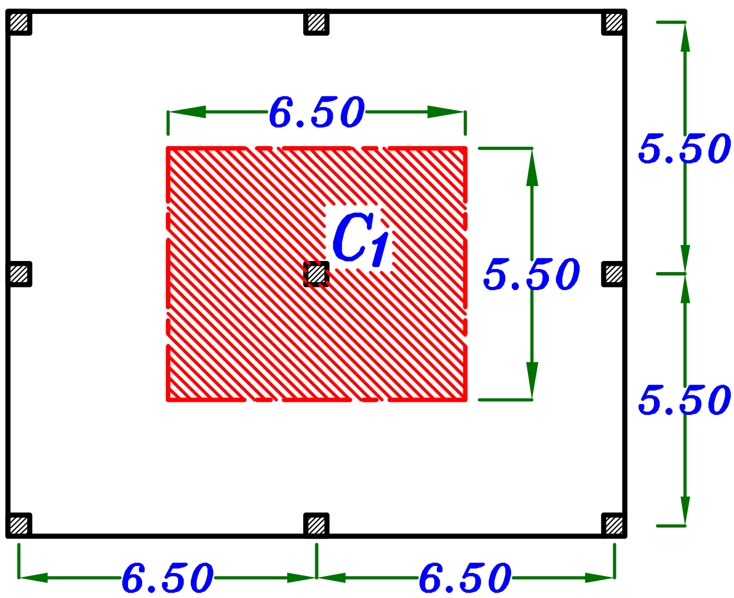
$$w_s = 1.4\,(0.22 * 25 + 1.50 + 1.50) + 1.6\,(2.0) = 15.10\ kN\backslash m^2$$

3- Check Punching on interior column C_1

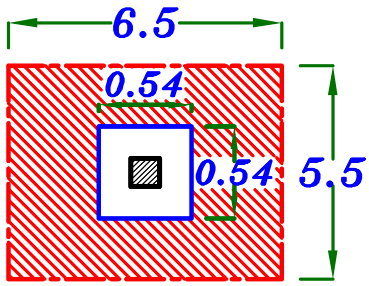
كل عمود يحمل مساحه

من $C.L.$ البلاطه

الى $C.L.$ البلاطه الاخرى



C_1 Interior Column.



$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$

$C + d = 0.35 + 0.19 = 0.54\ m$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 15.10\left[6.5 * 5.5 - 0.54 * 0.54\right] = 535.4\ kN$$

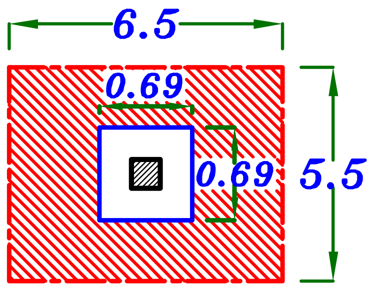
$$A_p = (b_o * d) = (4 * 540) * 190 = 410400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{535.4 * 10^3}{410400} * 1.15 = 1.50\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

$\boxed{q_{pu} > q_{pcu}}$ $\longrightarrow$ Unsafe punching
Increase dimensions of the column

C_1 Interior Column.



Take the Column $(500 * 500)$

$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$

$C + d = 0.50 + 0.19 = 0.69\ m$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 15.10\left[6.5 * 5.5 - 0.69 * 0.69\right] = 532.6\ kN$$

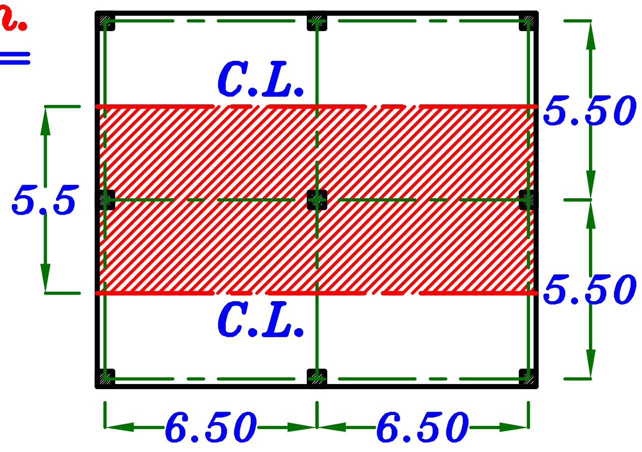
$$A_p = (b_o * d) = (4 * 690) * 190 = 524400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{532.6 * 10^3}{524400} * 1.15 = 1.17\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

$\boxed{q_{pu} < q_{pcu}}$ $\longrightarrow$ Safe Punching.

4-Frame at Long Direction.



$Span = L_1 = 6.5\ m$

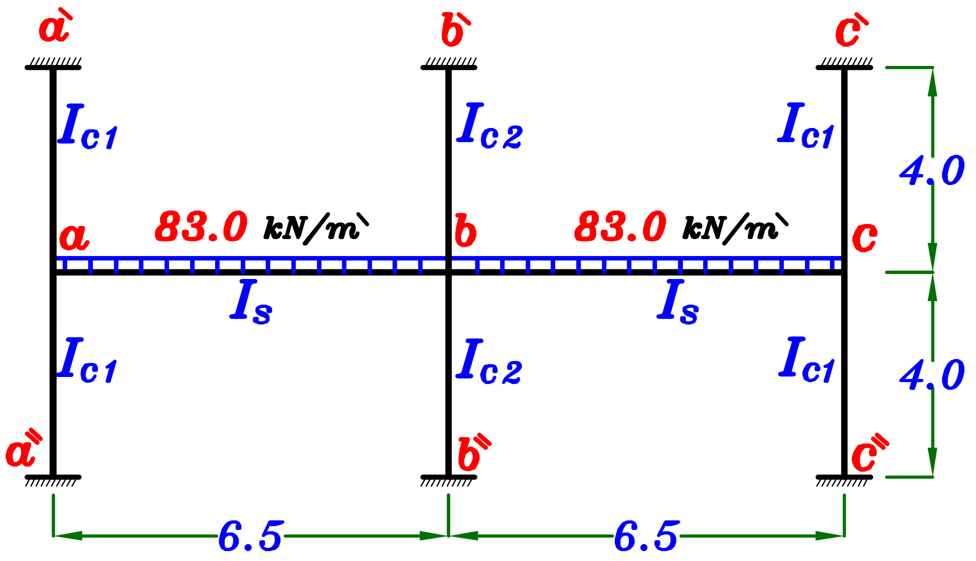
$Width = L_2 = 5.5\ m$

$$b_{C.S.} = \frac{L_2}{2} = 2.75\ m$$

$$b_{F.S.} = L_1 - \frac{L_2}{2} = 2.75\ m$$

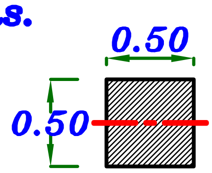
$$w = w_s * L_2 = 15.10 * 5.50 = 83.0\ kN/m$$

Use Moment Distribution.

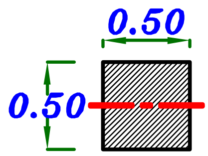


ⓐ Calculate Moment of Inertia For Slabs & Columns.

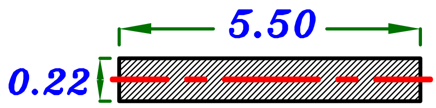
$$I_{c1} = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.50 * 0.50^3}{12} = 3.12 * 10^{-3}\ m^4$$



$$I_{c2} = 0.3 * \frac{b(t)^3}{12} = 0.3 * \frac{0.50 * 0.50^3}{12} = 1.56 * 10^{-3}\ m^4$$



$$I_s = \frac{L_2 * t_s^3}{12} = \frac{5.50 * 0.22^3}{12} = 4.88 * 10^{-3}\ m^4$$



ⓑ Calculate the stiffness For each member.

For Slabs. $$K_s = \frac{I_s}{L} = \frac{4.88 * 10^{-3}}{6.50} = 7.51 * 10^{-4}$$

For Columns. 1 $$K_c = \frac{I_{c1}}{h} = \frac{3.12 * 10^{-3}}{4.0} = 7.80 * 10^{-4}$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

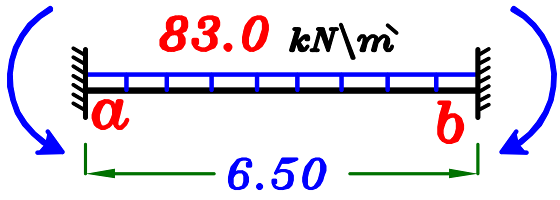
$$\Sigma K = K_s + 2K_c = 7.51 * 10^{-4} + 2 * 7.80 * 10^{-4} = 2.31 * 10^{-3}$$

$$D.F.(a\,a') = D.F.(a\,a'') = \frac{7.80 * 10^{-4}}{2.31 * 10^{-3}} = 0.337$$

$$D.F.(a\,b) = \frac{7.51 * 10^{-4}}{2.31 * 10^{-3}} = 0.326$$

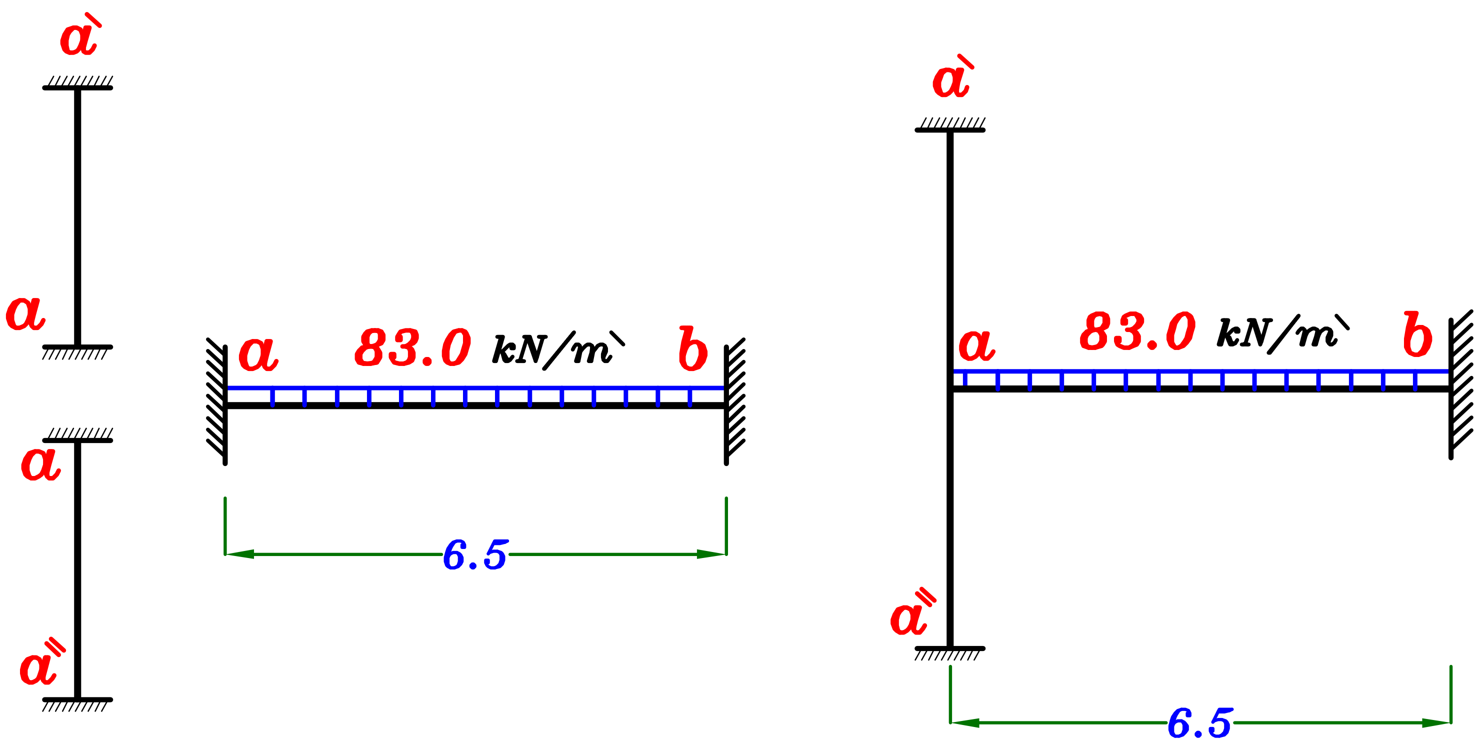
ⓓ Calculate Fixed End Moment For the Slab.

$$F.E.M._{(ab)} = -\frac{wL^2}{12} = -\frac{83.0 * 6.5^2}{12}$$

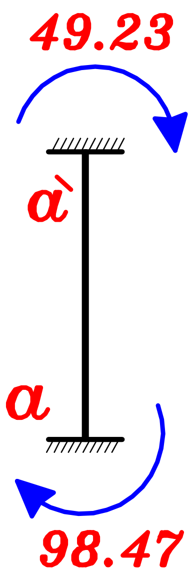


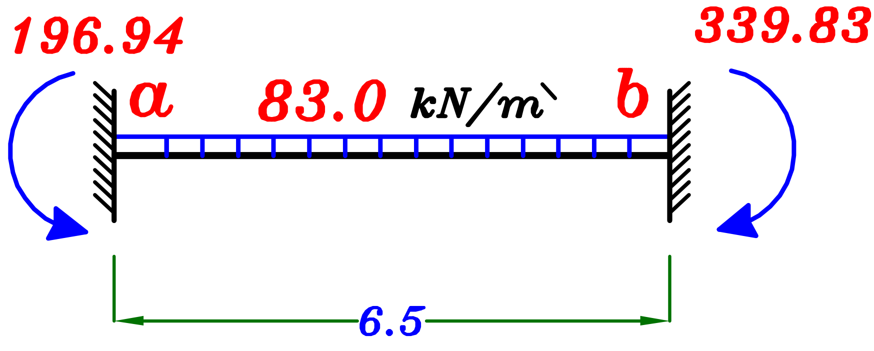
$$\boxed{F.E.M.\ (ab) = -\ 292.2\ kN.m}$$

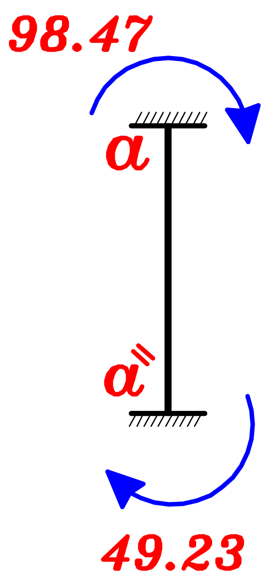
ⓔ Make the Table of Moment Distribution.

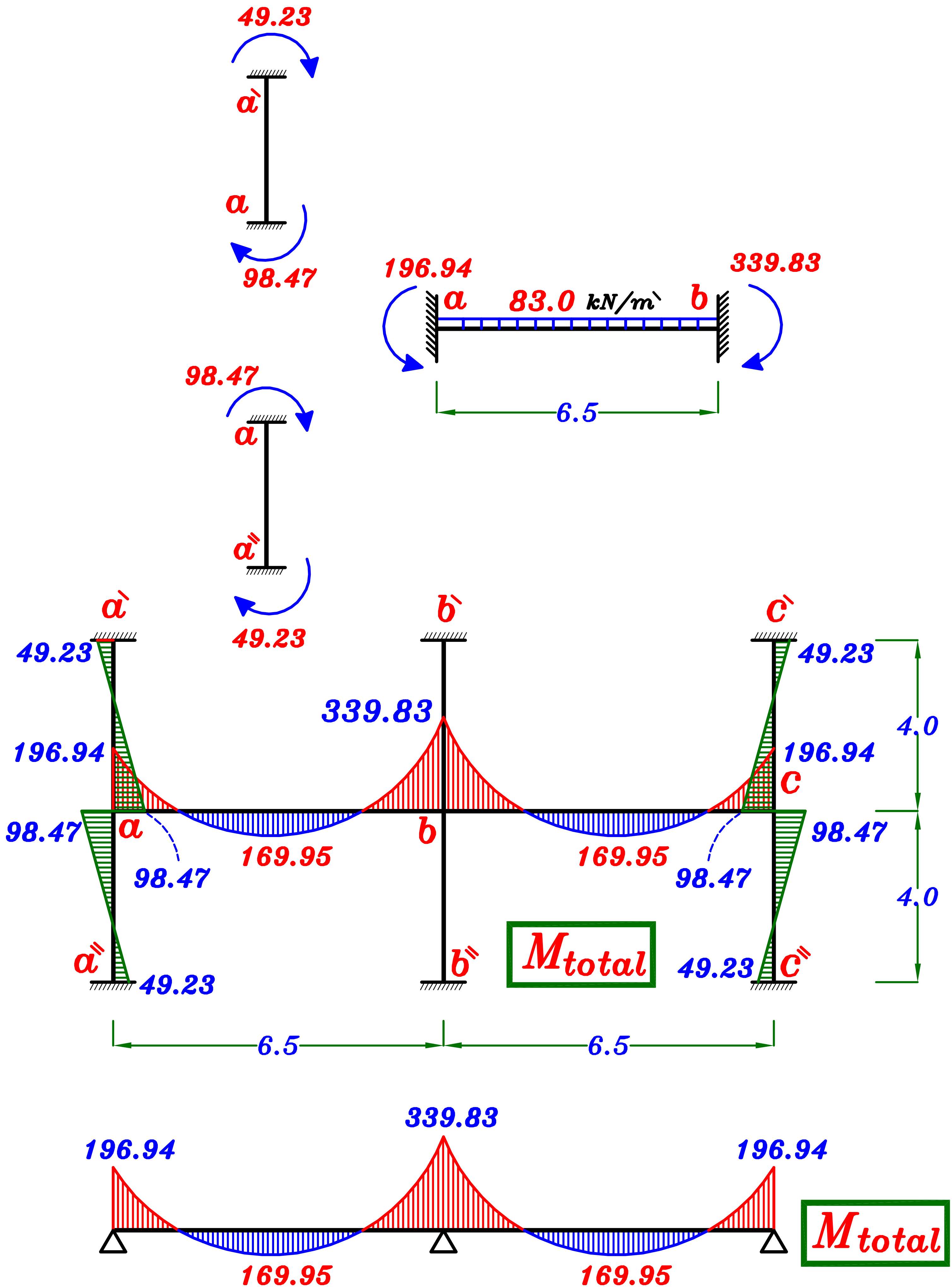


Joint	a\`	a			a\`\`	b
member	a\`-a	a-a\`	a-b	a-a\`\`	a\`\`-a	b-a
D.F.	0	0.337	0.326	0.337	0	0
F.E.M.	0	0	-292.2	0	0	+292.2
B.M.	0	+98.47	+95.26	+98.47	0	0
C.O.M.	+49.23	0	0	0	+49.23	+47.63
B.M.	0	0	0	0	0	0
M_F	+49.23	+98.47	-196.94	+98.47	+49.23	+339.83

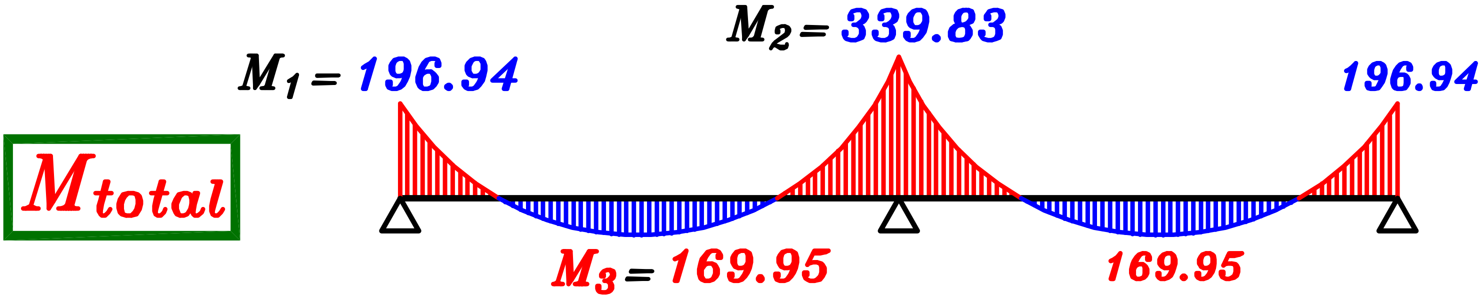


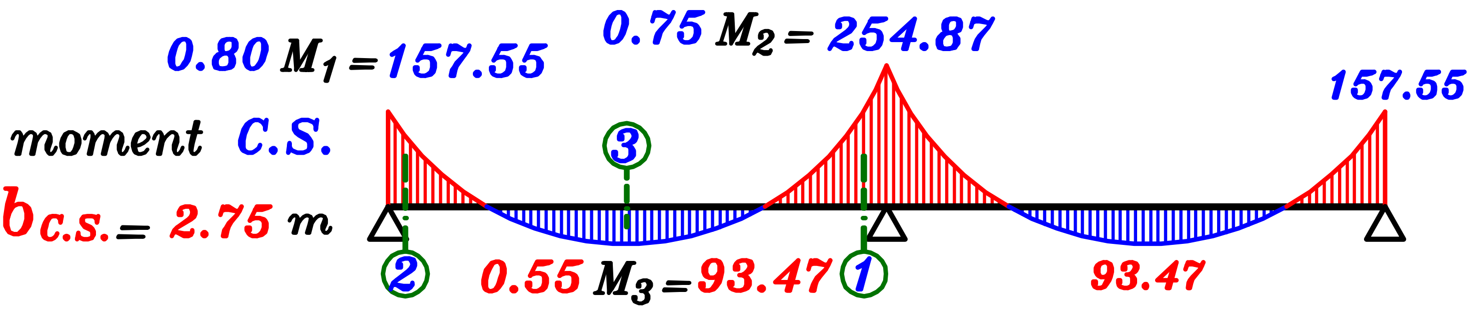


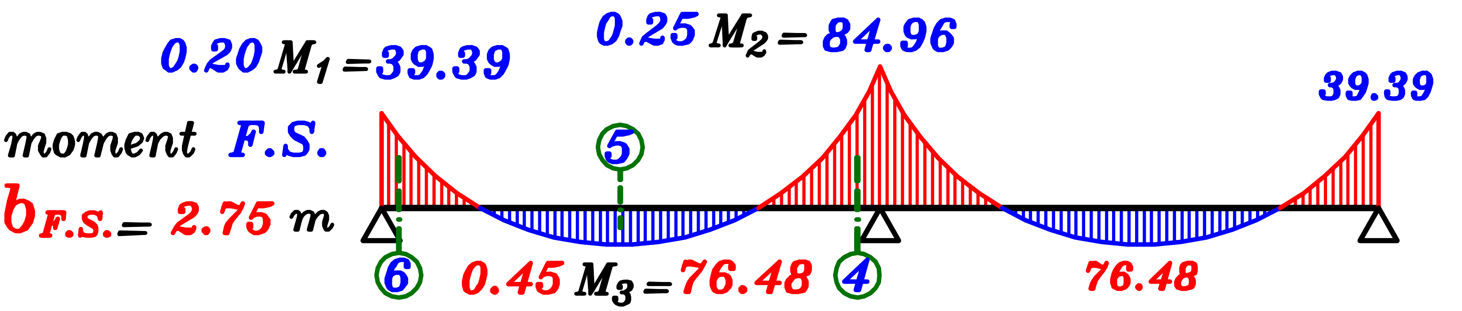



49.23
a`
a
98.47
196.94
339.83
a
83.0 kN/m`
b
6.5
98.47
a
a``
49.23
a`
b`
c`
49.23
49.23
339.83
196.94
196.94
c
4.0
98.47
a
b
98.47
169.95
169.95
98.47
98.47
4.0
a``
b``
49.23
49.23
c``
Mtotal
6.5
6.5
339.83
196.94
196.94
169.95
169.95
Mtotal

5- Distribute the moment of the Frame on Column Strip and Field Strip.



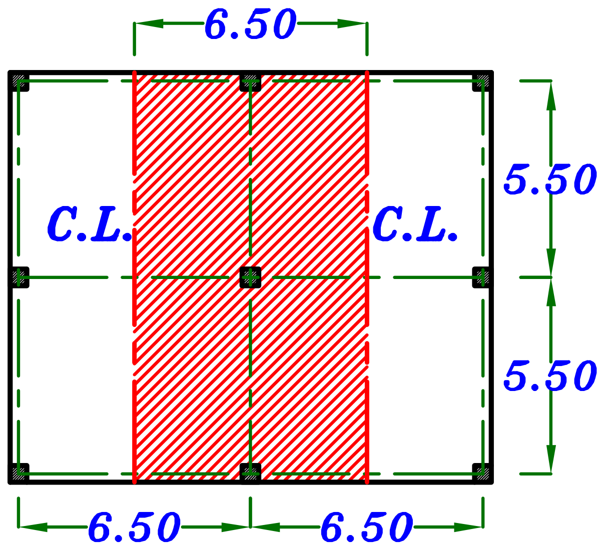




6- Design of sections. $d = t_s - 30$ mm

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	254.87	2750	190	3.12	0.754	4941	1796	8∅18\m
	2	157.55	2750	190	3.96	0.802	2872	1044	5∅18\m
	3	93.47	2750	190	5.15	0.826	1654	601	6∅12\m
Field Strip	4	84.96	2750	190	5.40	0.826	1503	546	5∅12\m
	5	76.48	2750	190	5.69	0.826	1353	492	5∅12\m
	6	39.39	2750	190	7.93	0.826	697	253	5∅12\m

3-Frame at Long Direction.



Span $= L_2 = 5.5$ m

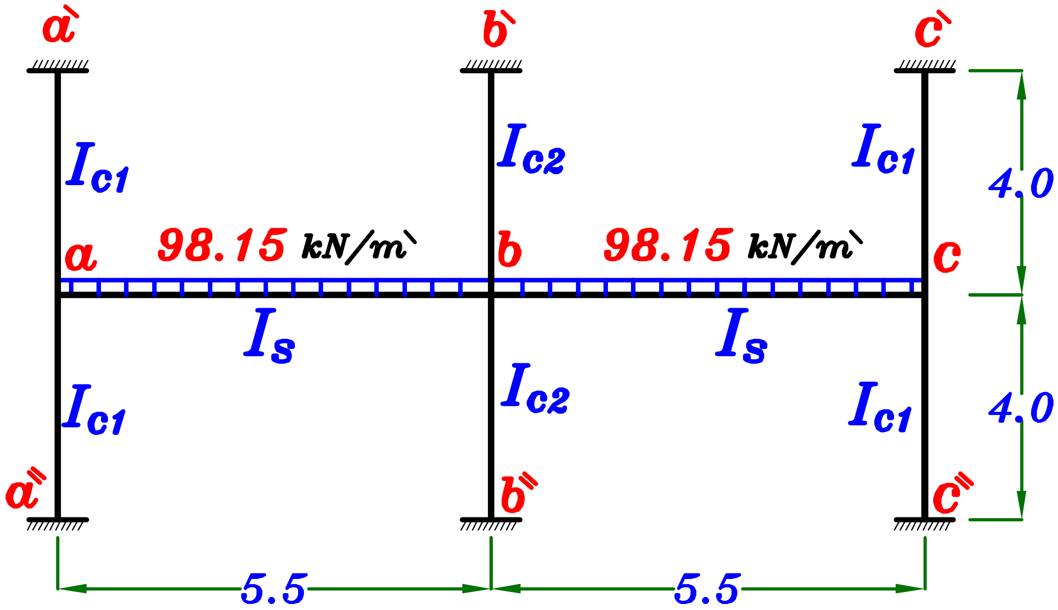
Width $= L_1 = 6.5$ m

$$b_{C.S.} = \frac{L_2}{2} = 2.75 \text{ m}$$

$$b_{F.S.} = L_1 - \frac{L_2}{2} = 3.75 \text{ m}$$

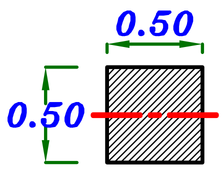
$$w = w_s * L_2 = 15.10 * 6.50 = 98.15 \text{ kN/m}$$

Use Moment Distribution.

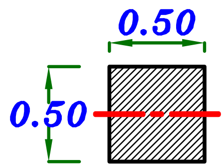


@ Calculate Moment of Inertia For Slabs & Columns.

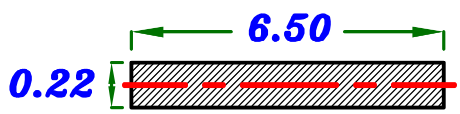
$$I_{c1} = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.50 * 0.50^3}{12} = 3.12 * 10^{-3} \text{ m}^4$$



$$I_{c2} = 0.3 * \frac{b(t)^3}{12} = 0.3 * \frac{0.50 * 0.50^3}{12} = 1.56 * 10^{-3} \text{ m}^4$$



$$I_S = \frac{L_2 * t_s^3}{12} = \frac{6.50 * 0.22^3}{12} = 5.76 * 10^{-3} \text{ m}^4$$



ⓑ Calculate the stiffness For each member.

For Slabs.

$$K_s = \frac{I_s}{L} = \frac{5.76 * 10^{-3}}{5.50} = 1.05 * 10^{-3}$$

For Columns. 1

$$K_{c1} = \frac{I_{c1}}{h} = \frac{3.12 * 10^{-3}}{4.0} = 7.80 * 10^{-4}$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

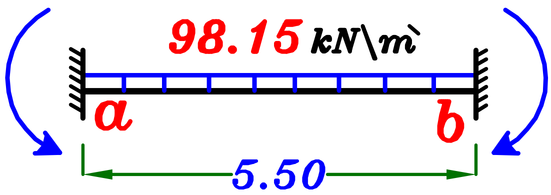
$$\Sigma K = K_s + 2K_c = 1.05 * 10^{-3} + 2 * 7.80 * 10^{-4} = 2.61 * 10^{-3}$$

$$D.F.(a\,a') = D.F.(a\,a'') = \frac{7.80 * 10^{-4}}{2.61 * 10^{-3}} = 0.299$$

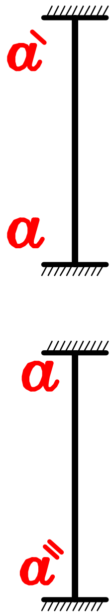
$$D.F.(a\,b) = \frac{1.05 * 10^{-3}}{2.61 * 10^{-3}} = 0.402$$

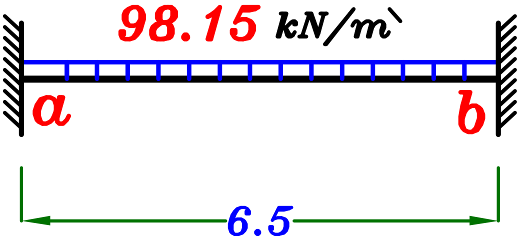
ⓓ Calculate Fixed End Moment For the Slab.

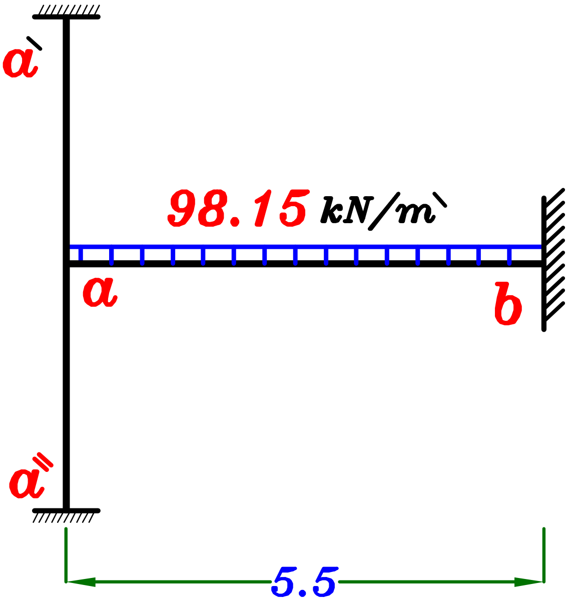
$$\underset{(ab)}{F.E.M.} = -\frac{wL^2}{12} = -\frac{98.15 * 5.5^2}{12}$$



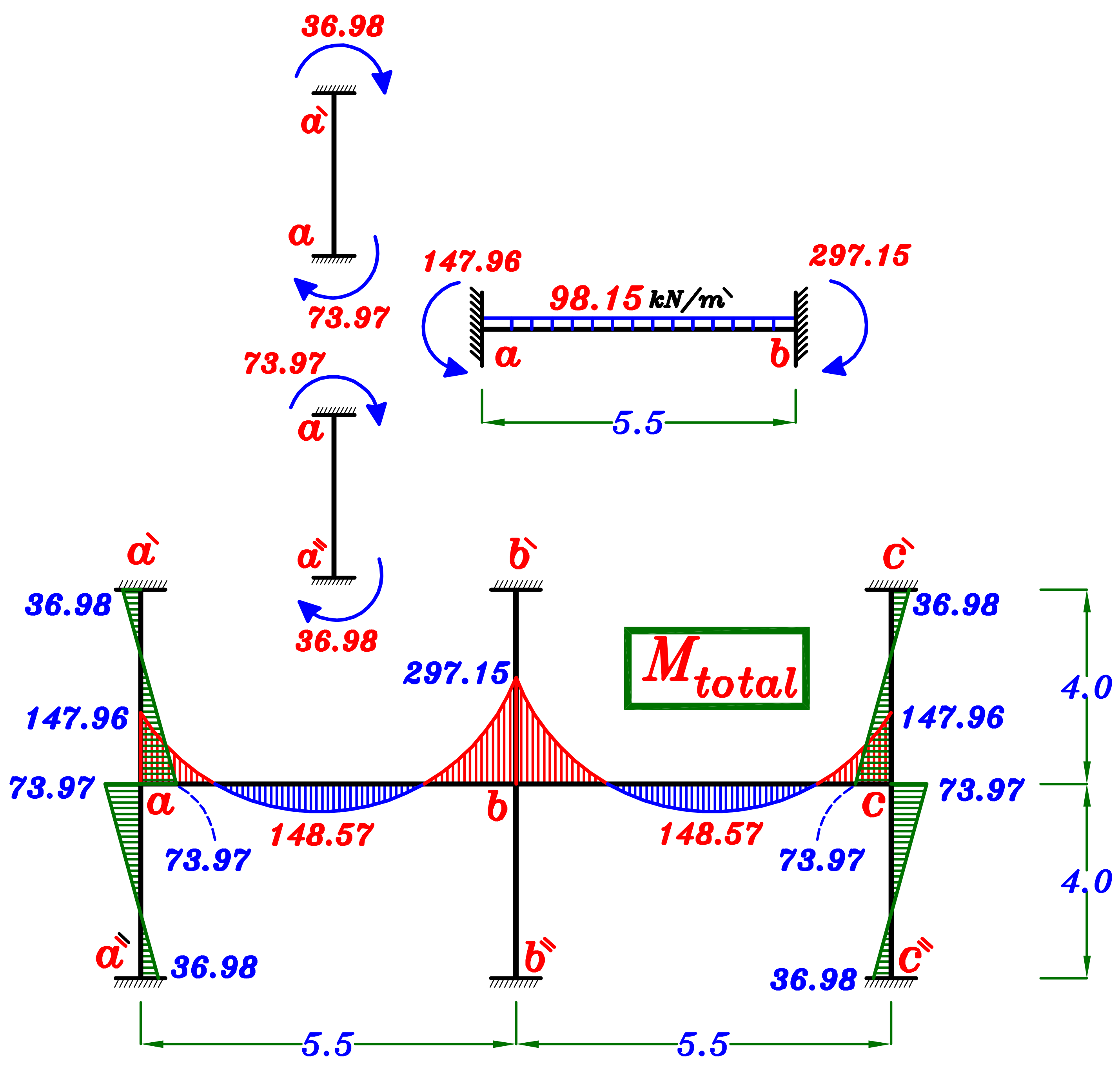
$$\boxed{F.E.M.\,(ab) = -247.42 \text{ kN.m}}$$







Joint	a`	a			a``	b
member	a`-a	a-a`	a-b	a-a``	a``-a	b-a
D.F.	0	0.299	0.402	0.299	0	0
F.E.M.	0	0	-247.42	0	0	+247.42
B.M.	0	+73.97	+99.46	+73.97	0	0
C.O.M.	+36.98	0	0	0	+36.98	+49.73
B.M.	0	0	0	0	0	0
M_F	+36.98	+73.97	-147.96	+73.97	+36.98	+297.15

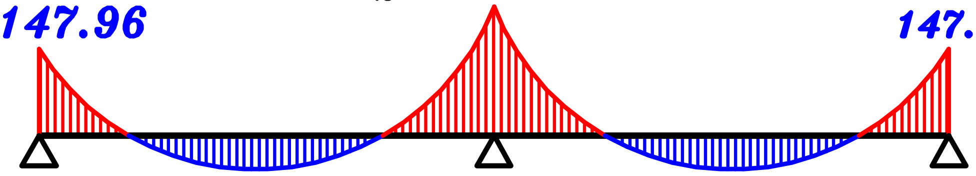


5-Distribute the moment of the Frame on Column Strip and Field Strip.

M_{total}

$M_2 = 297.15$

$M_1 = 147.96$ 147.96

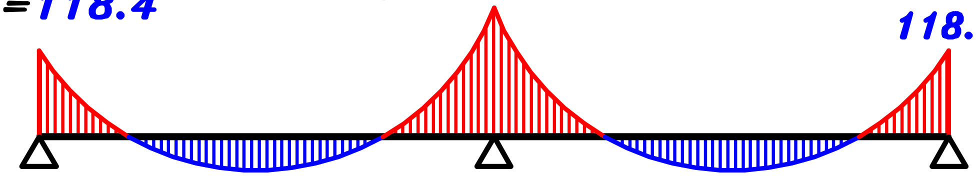


$M_3 = 148.57$ 148.57

$0.80\ M_1 = 118.4$ $0.75\ M_2 = 222.8$ 118.4

moment C.S.

$b_{C.S.} = 2.75$ m

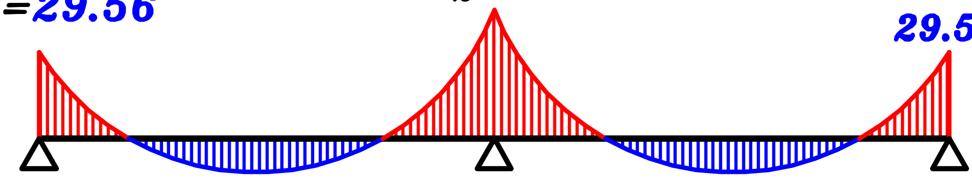


$0.55\ M_3 = 81.71$ 81.71

$0.20\ M_1 = 29.56$ $0.25\ M_2 = 74.35$ 29.56

moment F.S.

$b_{F.S.} = 3.75$ m



$0.45\ M_3 = 66.86$ 66.86

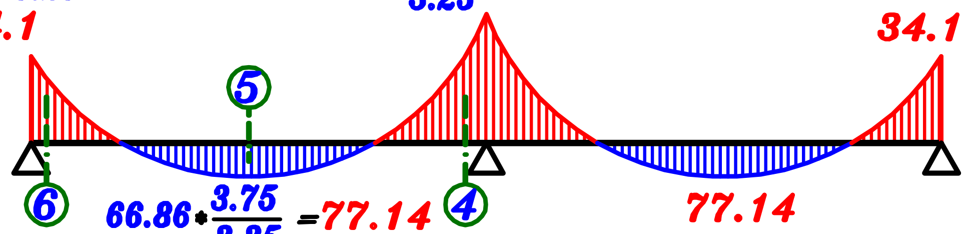
$$\text{Modification Factor} = \frac{L_1 - L_2 \backslash 2}{L_1 \backslash 2} = \frac{3.75}{3.25}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{3.75}{3.25}}$$

$29.56 * \frac{3.75}{3.25} = 34.1$ $74.35 * \frac{3.75}{3.25} = 85.79$ 34.1

moment F.S.

$b_{F.S.} = 3.75$ m



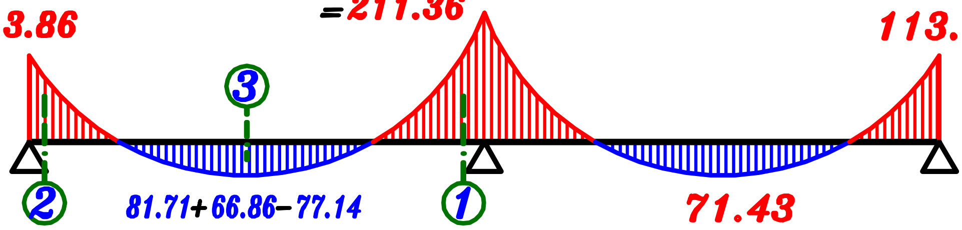
$66.86 * \frac{3.75}{3.25} = 77.14$ 77.14

$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

$118.4 + 29.56 - 34.1 = 113.86$ $222.8 + 74.35 - 85.79 = 211.36$ 113.86

moment C.S.

$b_{C.S.} = 2.75$ m



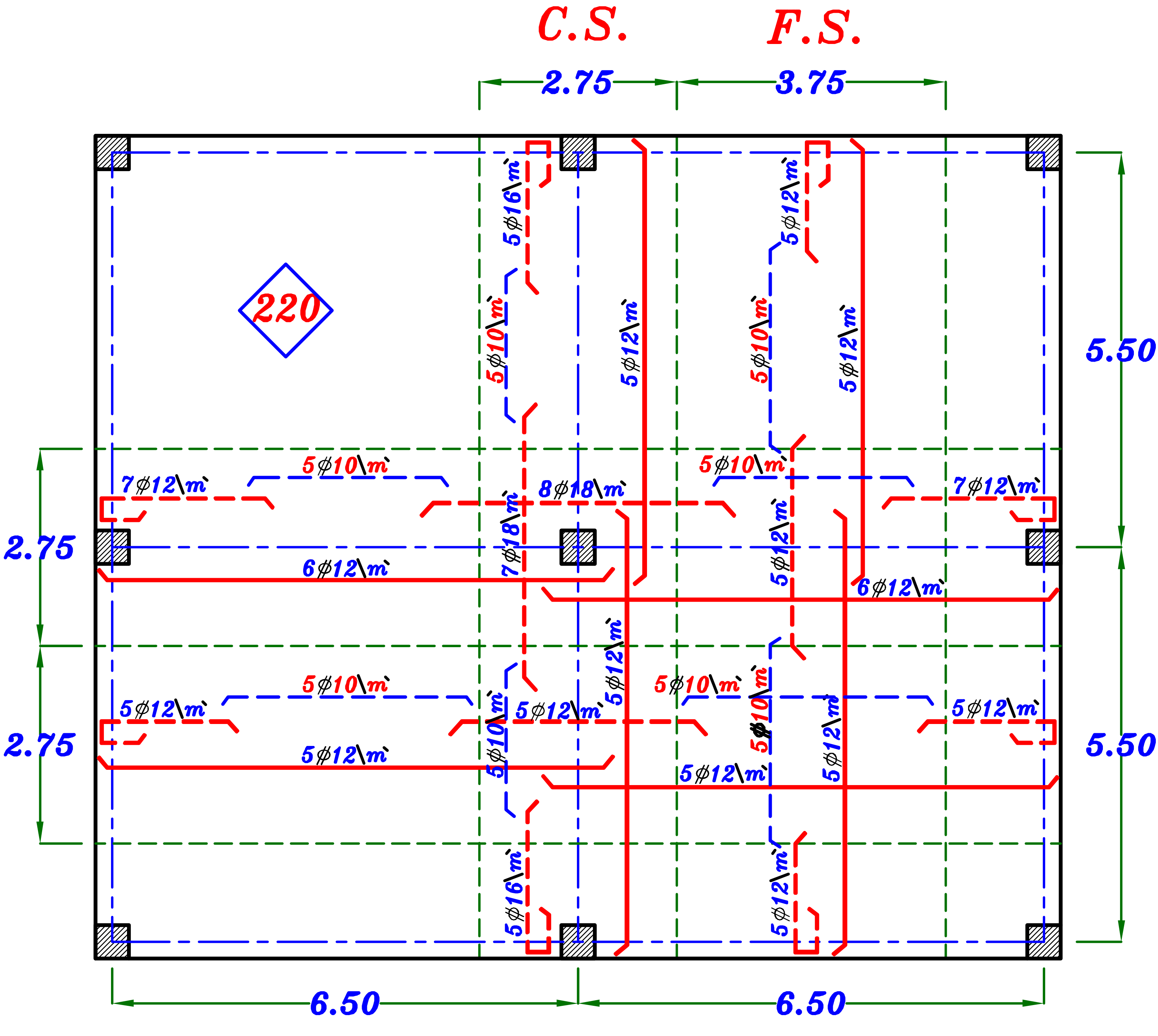
$81.71 + 66.86 - 77.14 = 71.43$ 71.43

6-Design of sections.

$d = t_s - 40$ mm

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm²/b)	As (mm²/m)	No. of bars/m`
Column Strip	1	211.36	2750	180	3.24	0.764	4269	1552	6ϕ18\m
	2	113.86	2750	180	4.42	0.816	2153	782	7ϕ12\m
	3	71.43	2750	180	5.58	0.826	1334	485	5ϕ12\m
Field Strip	4	85.79	3750	180	5.95	0.826	1603	427	5ϕ12\m
	5	77.14	3750	180	6.27	0.826	1441	384	5ϕ12\m
	6	34.10	3750	180	9.43	0.826	637	170	5ϕ12\m

7- Details of RFT.

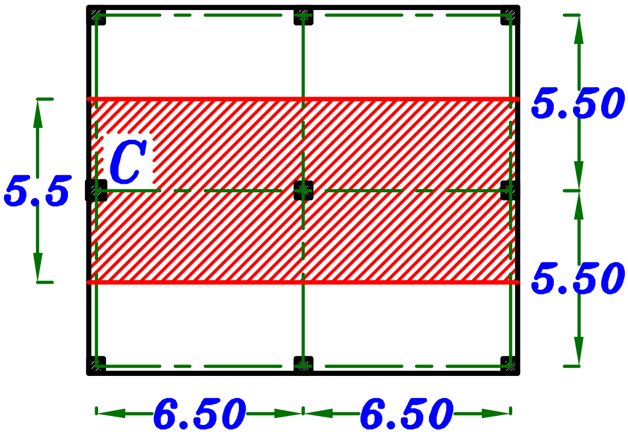


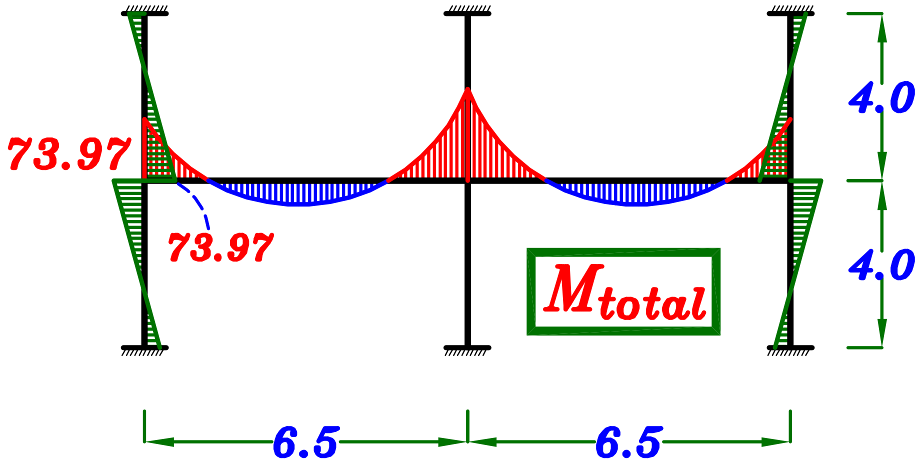
③ Design the column C at ground Floor.

Take the Column (500 * 500)

Frame analysis فى حاله حل البلاطه بطريقه

يتم أخذ العزم على العمود مباشره من ال Frame فى الاتجاه الطويل .





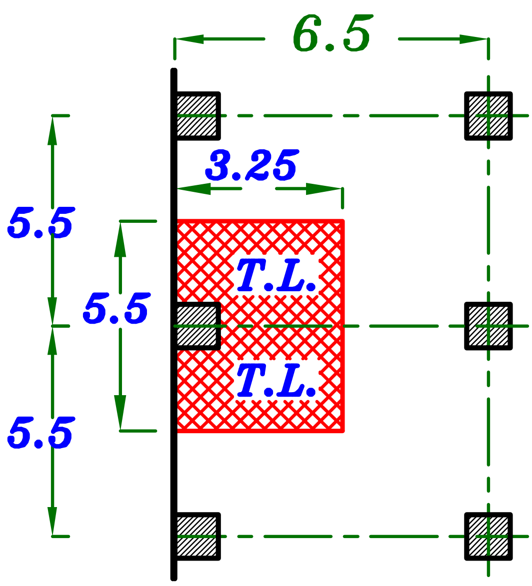
$$\boxed{M_{ext} = 98.47 \ kN.m}$$

$$P\backslash Floor = w_s * \left(\frac{L_1}{2} * L_2\right) * 1.1$$

$$P\backslash Floor = 15.10 * (3.25 * 5.5) * 1.1$$

$$= 296.9 \ kN$$

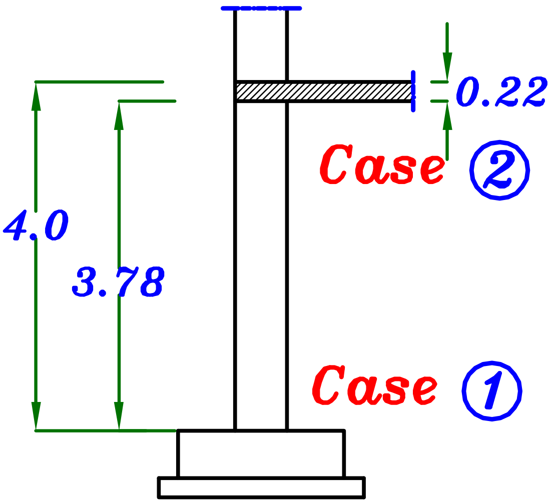
$$P_{(total)} = 296.9 * 5.0 = 1484.5 \ kN$$



$$\boxed{M_{ext} = 98.47 \ kN.m} \qquad \boxed{P_{total} = 1484.5 \ kN}$$

Check Buckling. In plane & out of plane.

العمود متماثل فى الاتجاهين

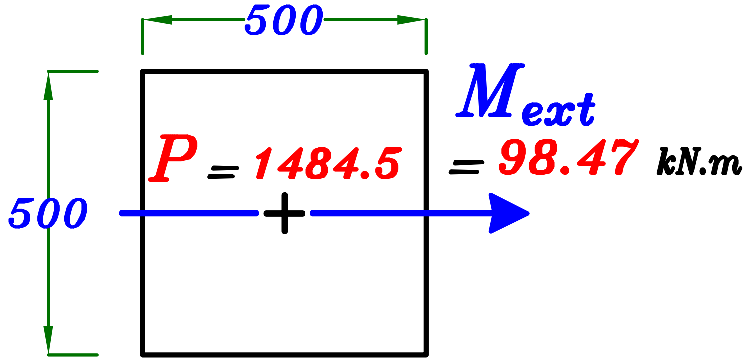


Upper Case ② }
Lower Case ① } $k = 1.3$

$H_o = 3.78\ m$

$$\lambda_b = \frac{1.3 * 3.78}{0.50} = 9.8 < 10$$

$\lambda_b < 10 \longrightarrow$ Short Column. $\longrightarrow$ NO M_{add}

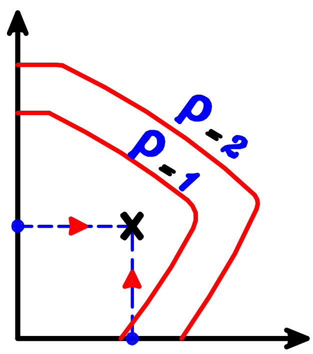


$$e = \frac{M}{P} = \frac{98.47}{1484.5} = 0.066\ m$$

$$\frac{e}{t} = \frac{0.066}{0.50} \simeq 0.132 \xrightarrow{use} I.D.$$

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use}$$ ECCS Design Aids Page 4–24

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{1484.5 * 10^3}{25 * 500 * 500} = 0.237 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{98.47 * 10^6}{25 * 500 * 500^2} = 0.031 \end{aligned}\right\} \rho = 1.0$$



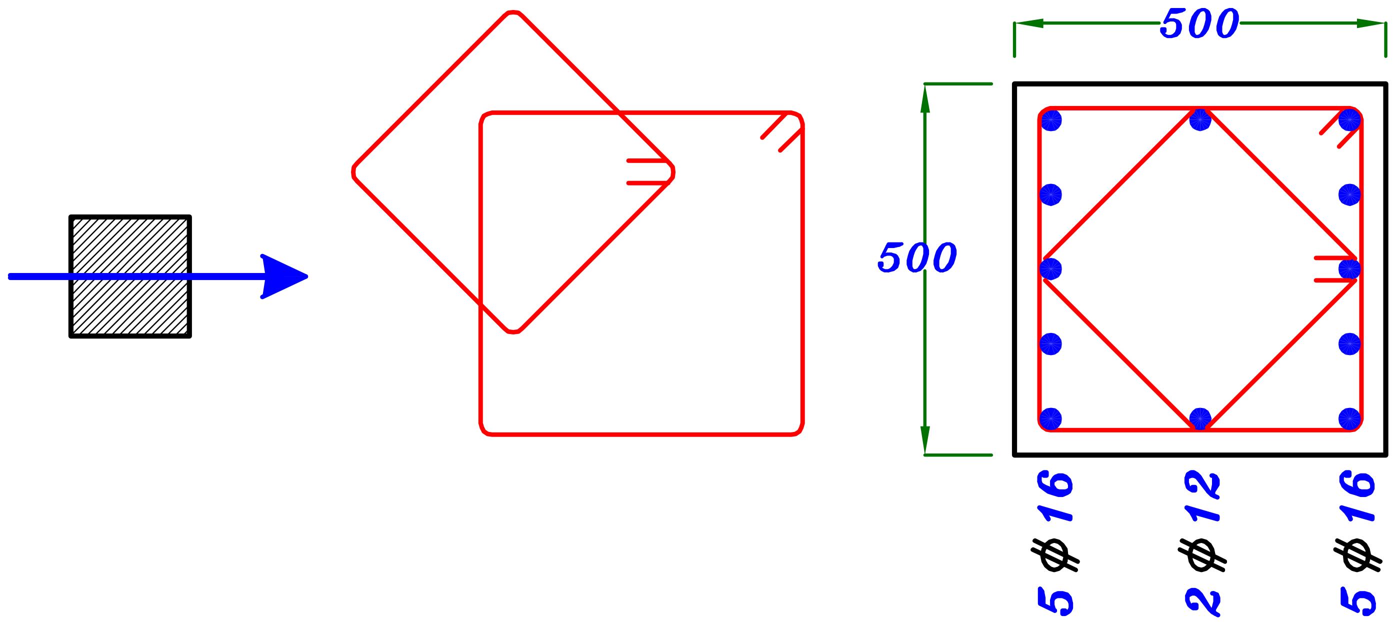
$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 500 * 500 = 625 \ mm^2$$

$$A_{s\,Total} = A_s + A_s` = 2 * 625 = 1250 \ mm^2$$

$$A_{s_{min}} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 500 * 500 = 2000 \ mm^2 > A_{s\,total}$$

Take $A_s = A_s` = \frac{A_{s\,min}}{2} = 1000 \ mm^2$ $\boxed{4 \phi 16}$



Example.

The given plan shows general layout of a **One Storey** Flat slab Floor. The column height 4.0 m

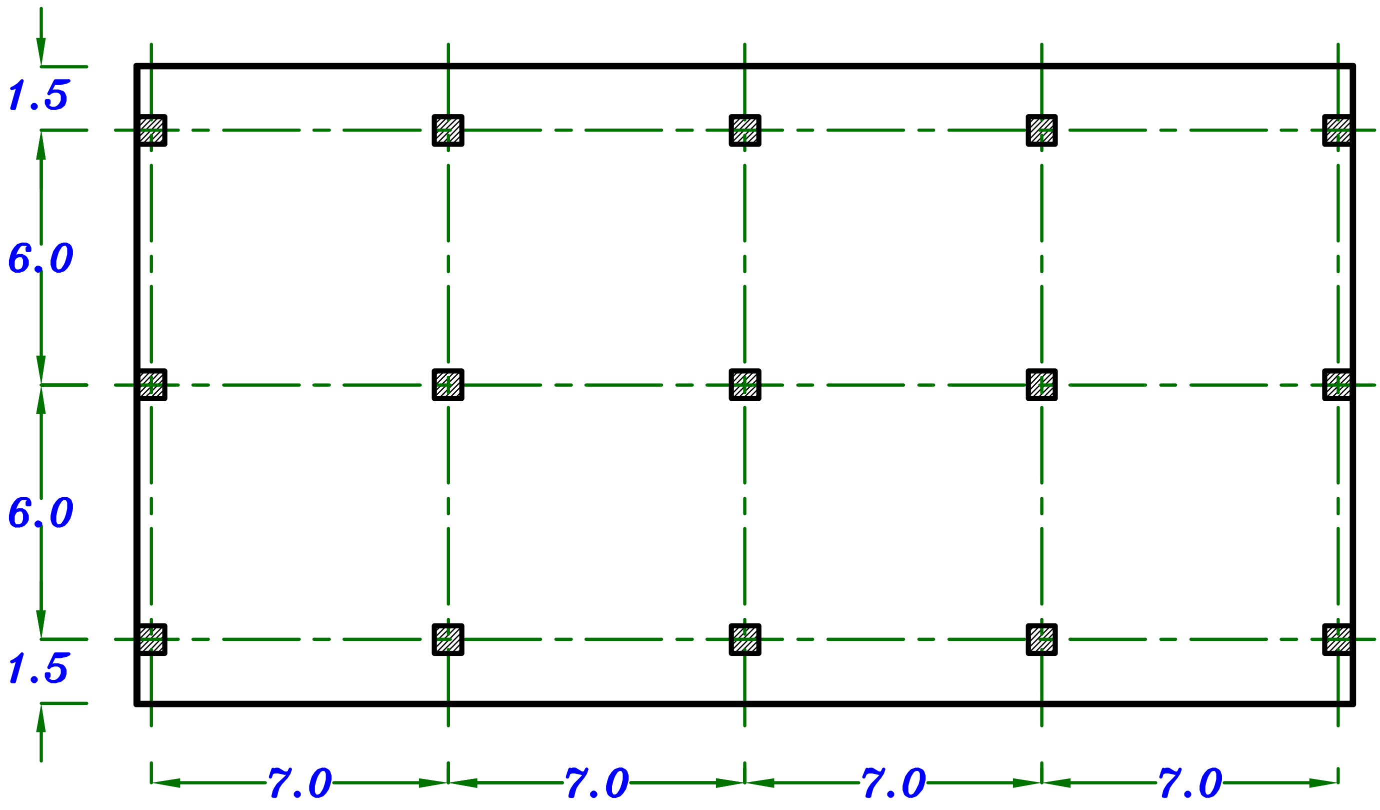
Data.

$F_{cu} = 30 \ N\backslash mm^2$, $F_y = 360 \ N\backslash mm^2$

$F.C. = 3.0 \ kN\backslash m^2$, $L.L. = 3.0 \ kN\backslash m^2$ لا توجد حوائط لانه دور أخير

Req.

1. Calculate the moments For both Field strip and column strip in both directions.
2. Design the sections of the slab and draw details of reinforcement in plan.



لحل هذه البلاطه المفروض ان يحل الاتجاهان بال **Frame analysis method**
لان يوجد اتجاه منهم عدد الباكيات فيه اقل من ٣ باكيات .
لكن للتسهيل ممكن حل الاتجاه الذى فيه عدد الباكيات ٢ فقط بال **Frame analysis**
و الاتجاه الاخر الذى عدد الباكيات فيه اكثر من ٣ باكيات بال **Empirical method**

Solution.

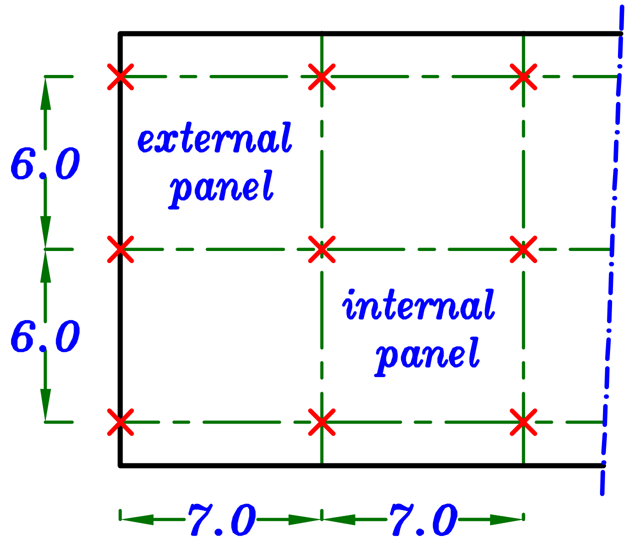
1- Concrete Dimensions.

Column dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{4000}{15} = 266.6\ mm \\ \dfrac{L_1}{20} = \dfrac{7000}{20} = 350\ mm \end{cases}$$

$$\boxed{b_{Col.} = 350\ mm}$$

$(350 * 350)$



Slab Thikness.

$L_1 = 7.0\ m$

External panel $t_s = \dfrac{L_1}{32} = \dfrac{7000}{32} = 218.7\ mm$

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7000}{36} = 194.4\ mm$

$$\boxed{t_s = 220\ mm}$$

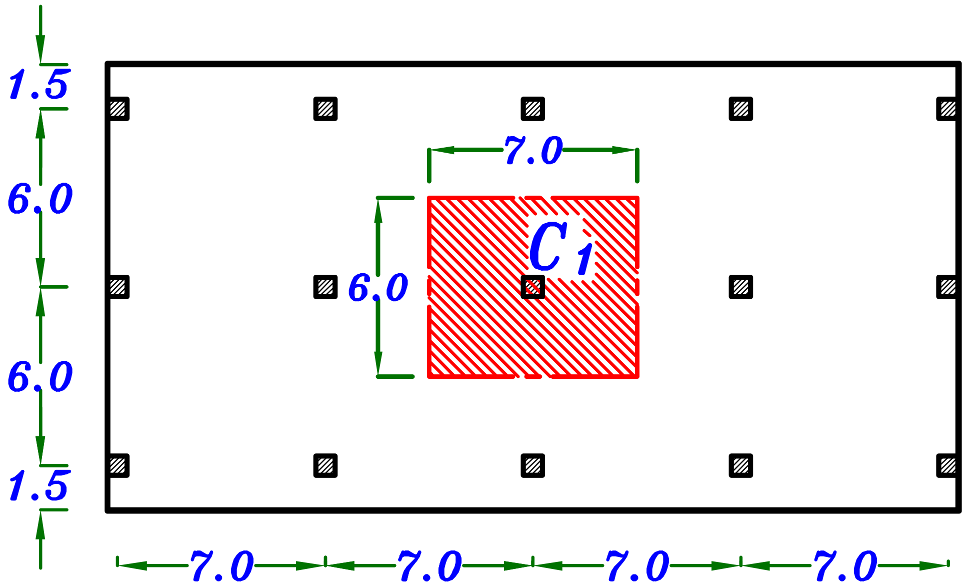
2- Loads on the Slab.

$$w_{s\,U.L.} = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)$$

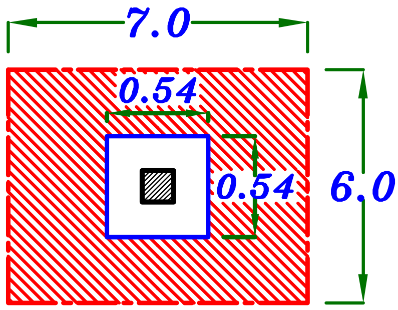
$$w_{s\,U.L.} = 1.4\,(0.22 * 25 + 3.0) + 1.6\,(3.0) = 16.70\ kN\backslash m^2$$

3 – Check Punching on interior column C_1

كل عمود يحمل مساحه
من البلاطه C.L.
الى البلاطه الاخرى C.L.



C_1 Interior Column.



$$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$$

$$C + d = 0.35 + 0.19 = 0.54\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 16.70\left[7.0 * 6.0 - 0.54 * 0.54\right] = 696.5\ kN$$

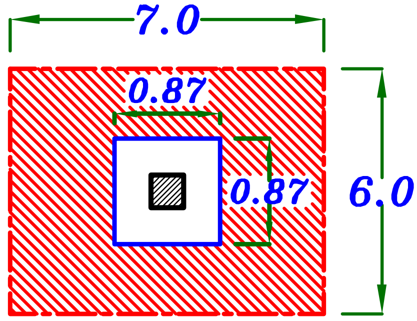
$$A_p = (b_o * d) = (4 * 540) * 190 = 410400\ \ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{696.5 * 10^3}{410400} * 1.15 = 1.95\ \ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\delta_c}} = 0.316\sqrt{\frac{30}{1.5}} = 1.41\ \ N/mm^2$$

$$\boxed{q_{pu} > q_{pcu}}$$ $\longrightarrow$ Unsafe punching
Increase dimensions of the column

C_1 Interior Column.



Take the Column (700 * 700)

$d = t_s - 30\,mm = 200 - 30 = 170\ mm = 0.17\ m$

$C + d = 0.70 + 0.17 = 0.87\ m$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1 + d)(C_2 + d) \right]$$

$$Q_{pu} = 16.70 \left[7.0 * 6.0 - 0.87 * 0.87 \right] = 688.7\ kN$$

$$A_p = (b_o * d) = (4 * 870) * 170 = 591600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{688.7 * 10^3}{591600} * 1.15 = 1.33\ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$$

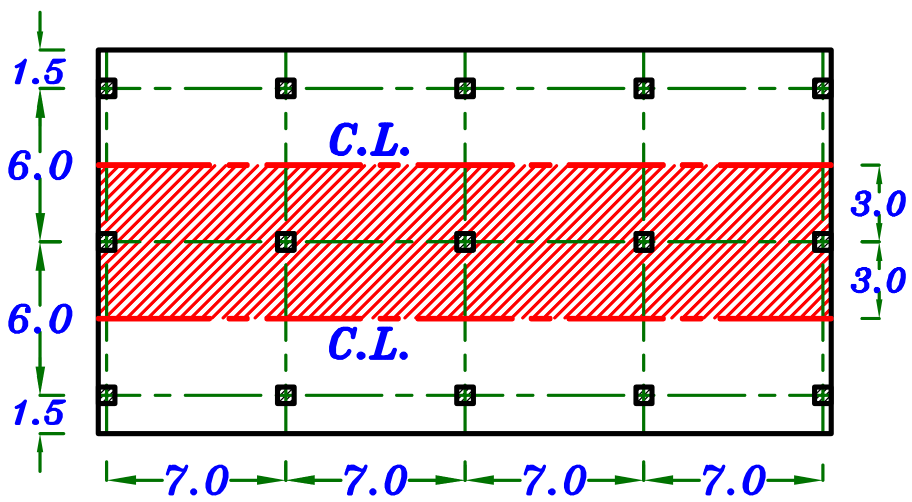
$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.

4- Moment at Long Direction.

Use Empirical Method.

فى الاتجاه الطويل نختار حساب العزوم بطريقه Empirical Method لانها أسهل .
و خاصه أن عدد البواكى فى هذا الاتجاه أكثر من ٣ بواكى .

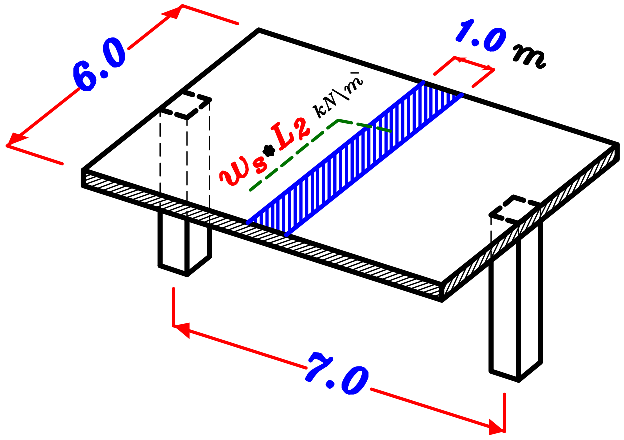
يفضل اختيار C.L. من المنتصف
و ليس عند الـCantilever



Total moment on the panel.

Span $= 7.0\ m$

Width $= 6.0\ m$

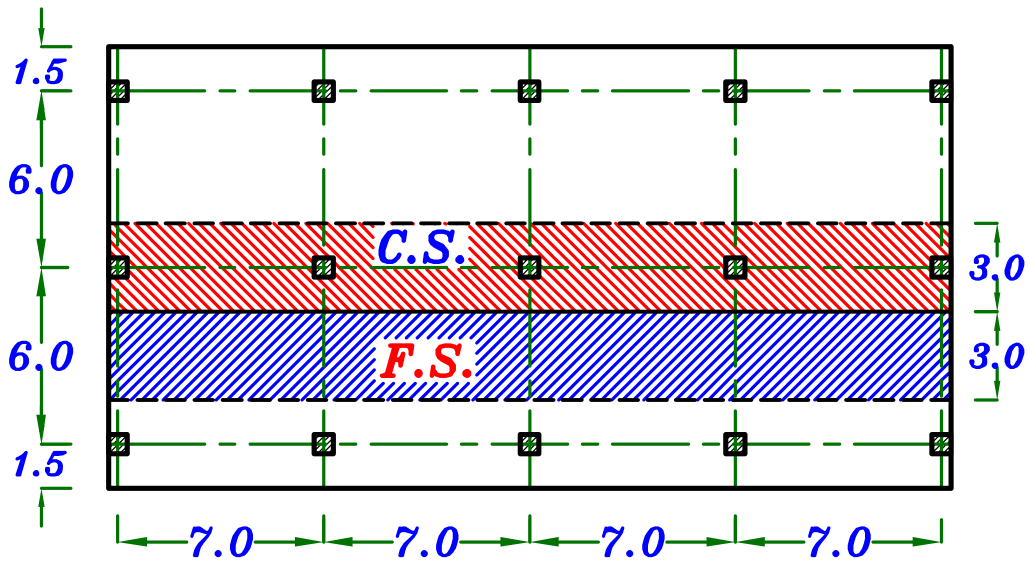


Moment in Long Direction.

$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8} = \frac{(16.70 * 6.0)\left(7.0 - \frac{2}{3} * 0.70\right)^2}{8}$$

$$\boxed{M_o = 534.6\ kN.m}$$ Long Direction

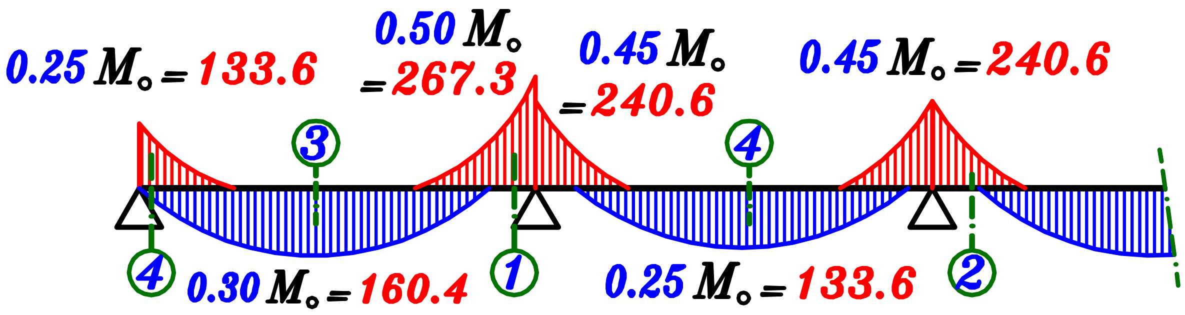
5- Distribute the B.M. (M_o) on C.S. & F.S.



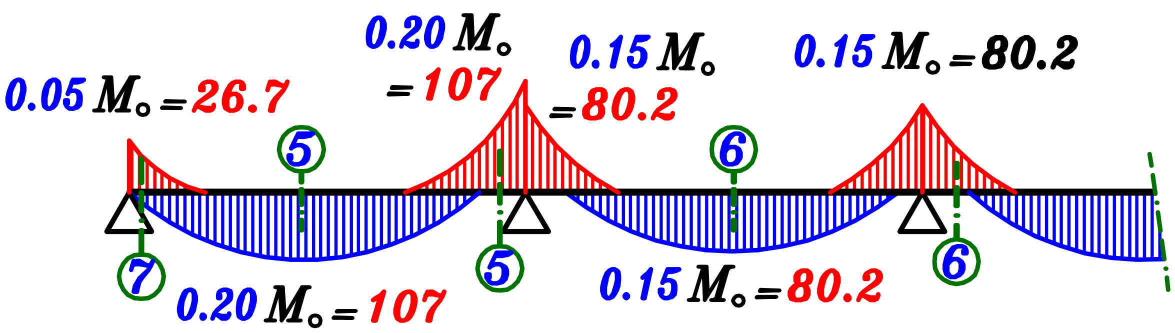
Column Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$

Field Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$

$M_o = 534.6$ kN.m **Long Direction**



Column Strip



Field Strip

6-Design of sections.

$d = t_s - 30\ mm = 220 - 30 = 190\ mm$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	267.3	3000	190	3.48	0.780	5010	1670	7ϕ18\m
	2	240.6	3000	190	3.67	0.788	4464	1488	6ϕ18\m
	3	160.4	3000	190	4.50	0.818	2866	955	5ϕ16\m
	4	133.6	3000	190	4.90	0.826	2364	788	7ϕ12\m
Field Strip	5	107	3000	190	5.51	0.826	1894	631	6ϕ12\m
	6	80.2	3000	190	6.36	0.826	1419	473	5ϕ12\m
	7	26.7	3000	190	11.03	0.826	472	157	5ϕ12\m

4-Moment at Short Direction.

Use Frame analysis Method.

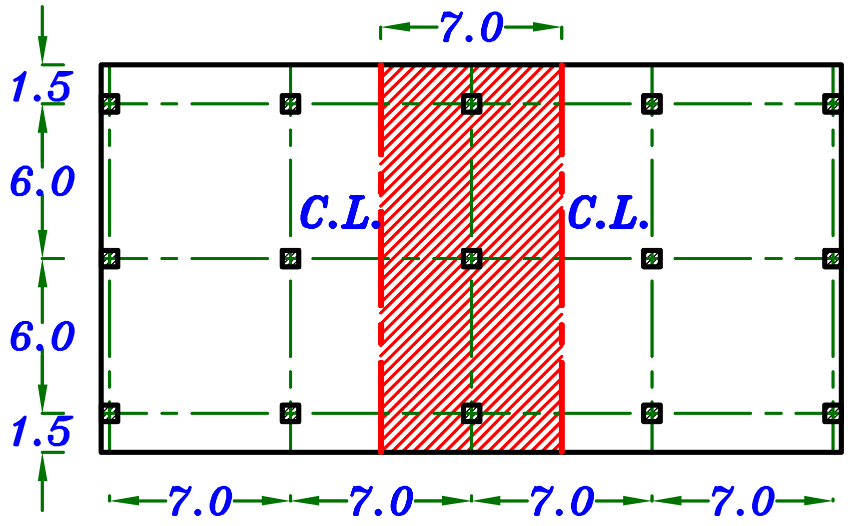
فى الاتجاه القصير نختار حساب العزوم بطريقه **Frame analysis method**
لانه لن ينفع استخدام طريقه ال **Empirical** لان عدد البواكى أقل من ٣ بواكى .

$Span = L_2 = 6.0 \ m$

$Width = L_1 = 7.0 \ m$

$b_{C.S.} = \frac{L_2}{2} = 3.0 \ m$

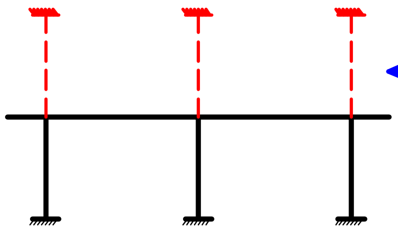
$b_{F.S.} = L_1 - \frac{L_2}{2} = 4.0 \ m$

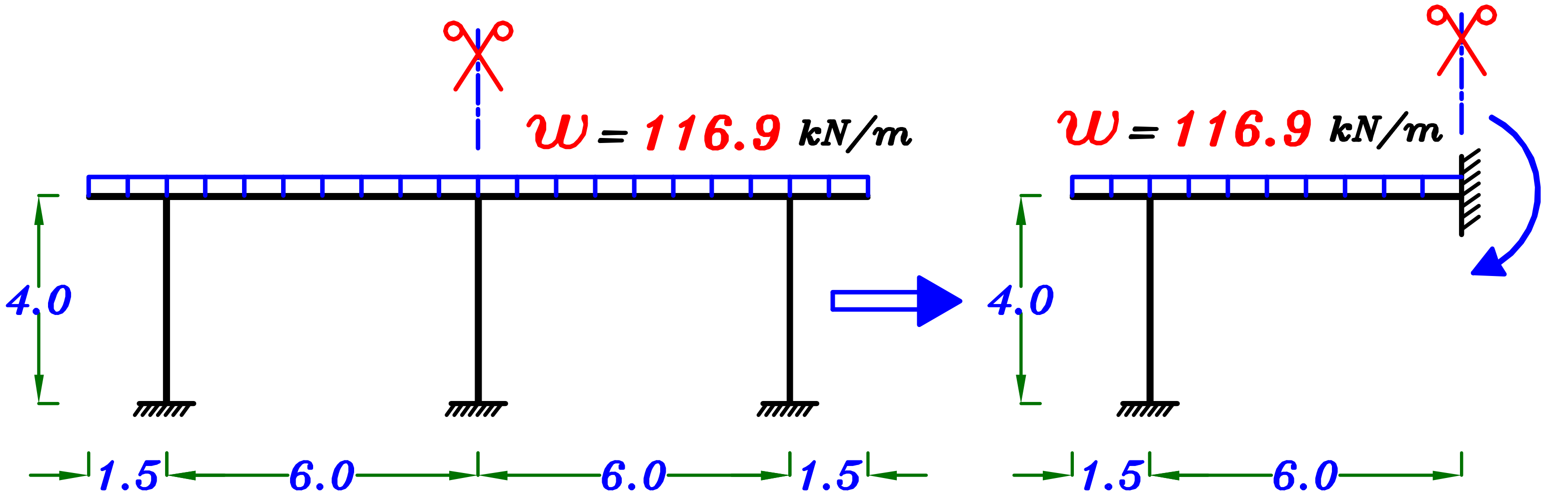


$$w = w_S * L_2 = 16.70 * 7.0 = 116.9 \ kN/m$$

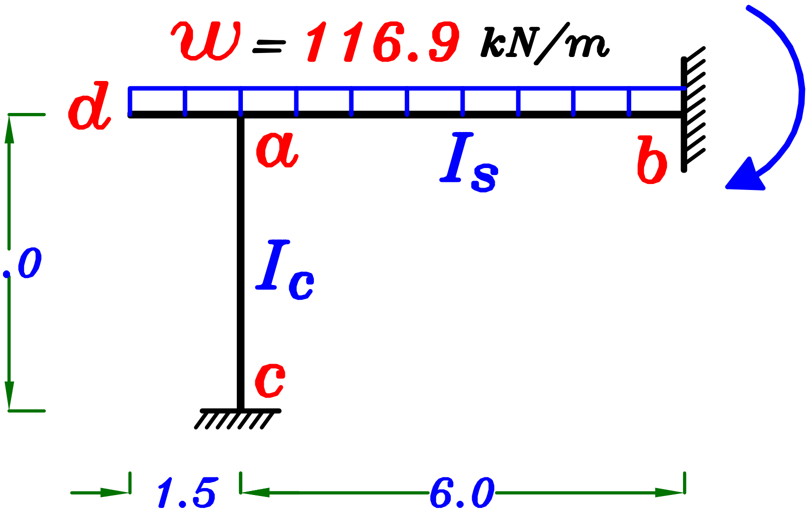
لاحظ فى المسأله أن السقف **One Storey** أى دور واحد لذا لا توجد أعمده

لذا لا توجد أعمده أعلى البلاطه .



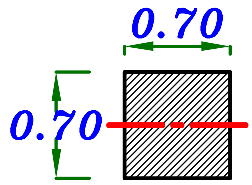


لان ال **Frame** يعتبر **symmetric** فممكن حل نصفه فقط.

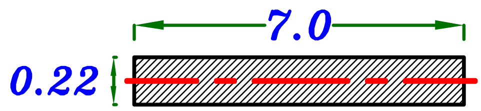


ⓐ Calculate Moment of Inertia For Slabs & Columns.

$$I_c = 0.60 * \frac{b(t)^3}{12} = 0.60 * \frac{0.70 * 0.70^3}{12} = 12 * 10^{-3} \ m^4$$



$$I_S = \frac{L_1 * t_s^3}{12} = \frac{7.0 * 0.22^3}{12} = 6.21 * 10^{-3} \ m^4$$



ⓑ Calculate the stiffness For each member.

For Slab.

$$K_{ab} = \frac{I_s}{L} = \frac{6.21 * 10^{-3}}{6.0} = 1.035 * 10^{-3}$$

لم نضرب K_{ab} فى $\frac{1}{2}$ لان ال member ليس symmetric لان $(M_{ab} \neq M_{ba})$

For Column.

$$K_{ac} = \frac{I_c}{h} = \frac{12 * 10^{-3}}{4.0} = 3.0 * 10^{-3}$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

$$\Sigma K = K_{ab} + K_{ac} = 1.035 * 10^{-3} + 3.0 * 10^{-3} = 4.035 * 10^{-3}$$

$$D.F.(ab) = \frac{K_{ab}}{\Sigma K} = \frac{1.035 * 10^{-3}}{4.035 * 10^{-3}} = 0.256$$

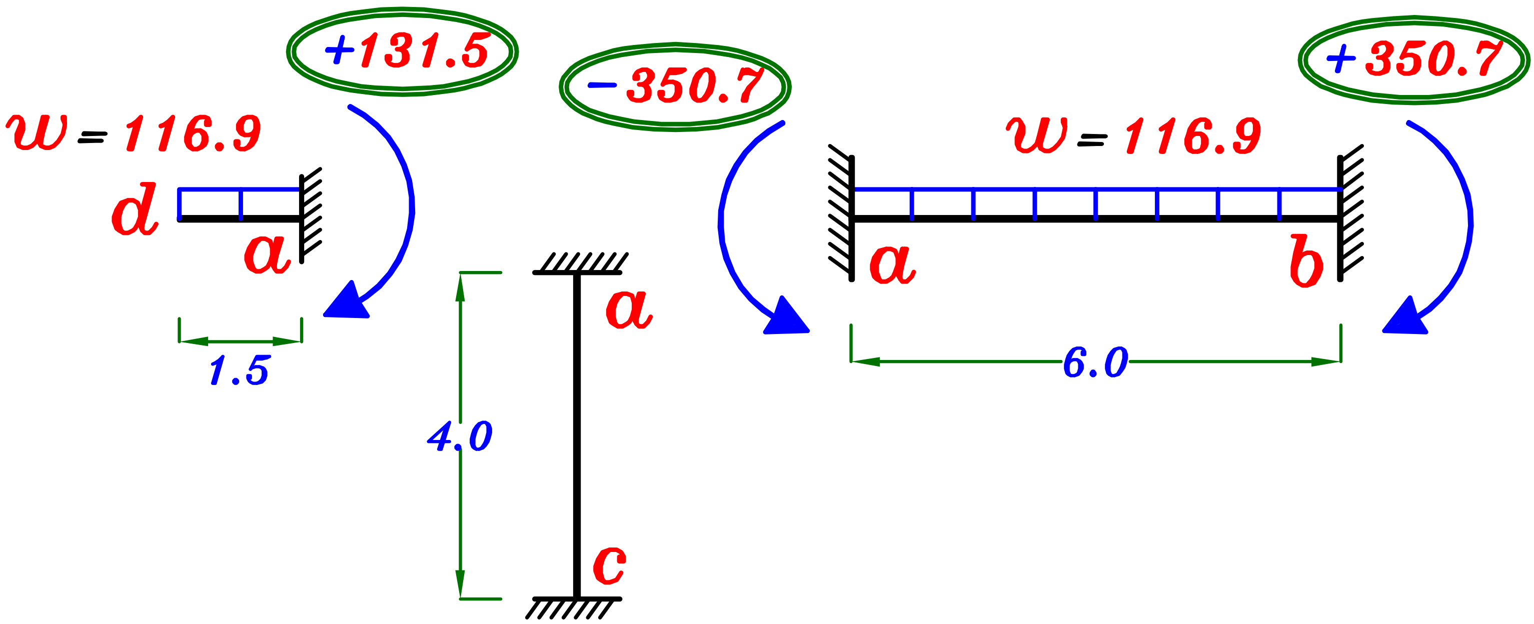
$$D.F.(ac) = \frac{K_{ac}}{\Sigma K} = \frac{3.0 * 10^{-3}}{4.035 * 10^{-3}} = 0.744$$

ⓓ Calculate Fixed End Moment For the Slab.

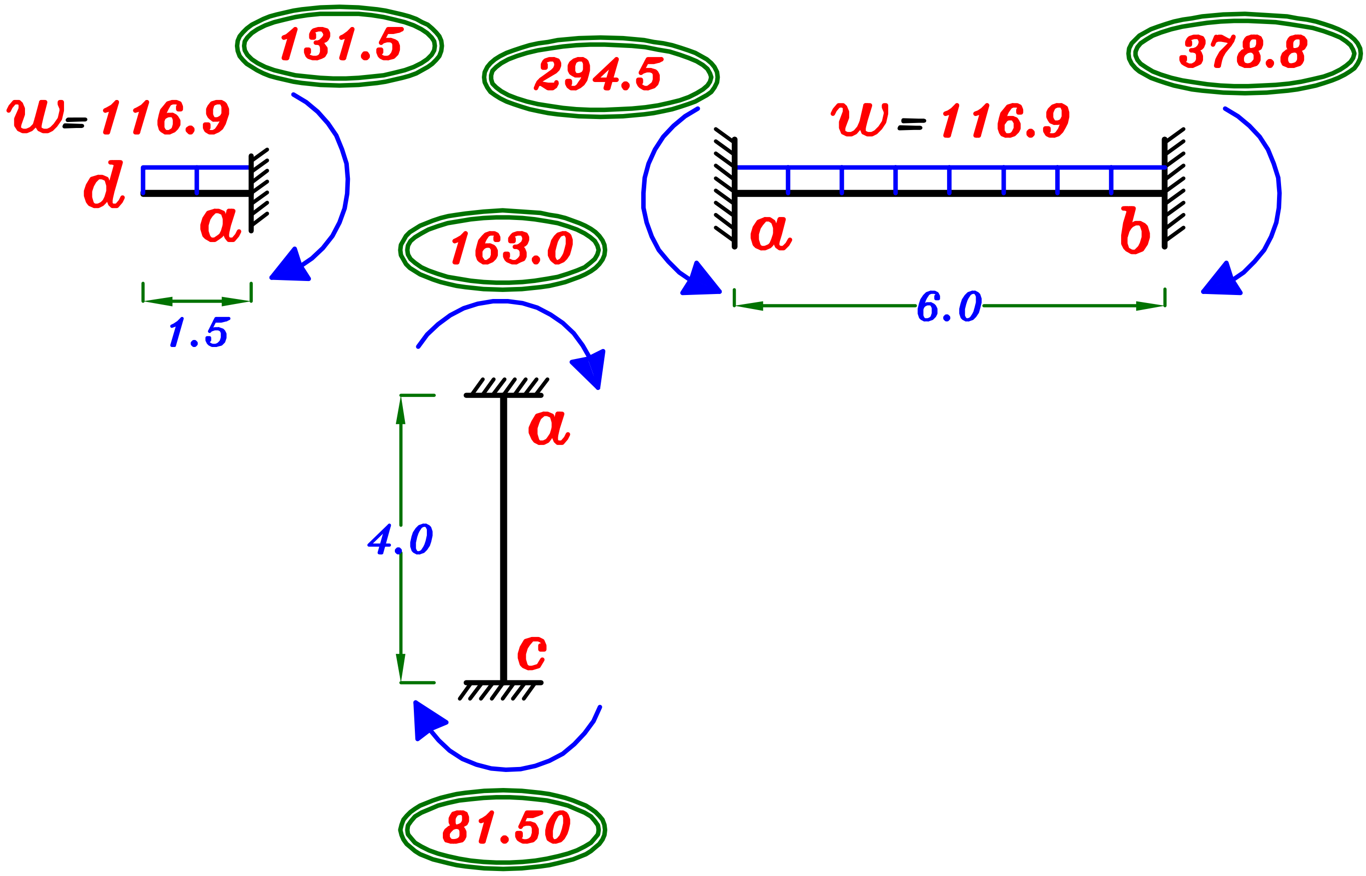
$$F.E.M.(ab) = -\frac{wL^2}{12} = -\frac{116.9 * 6.0^2}{12} = -350.7 \ kN.m.$$

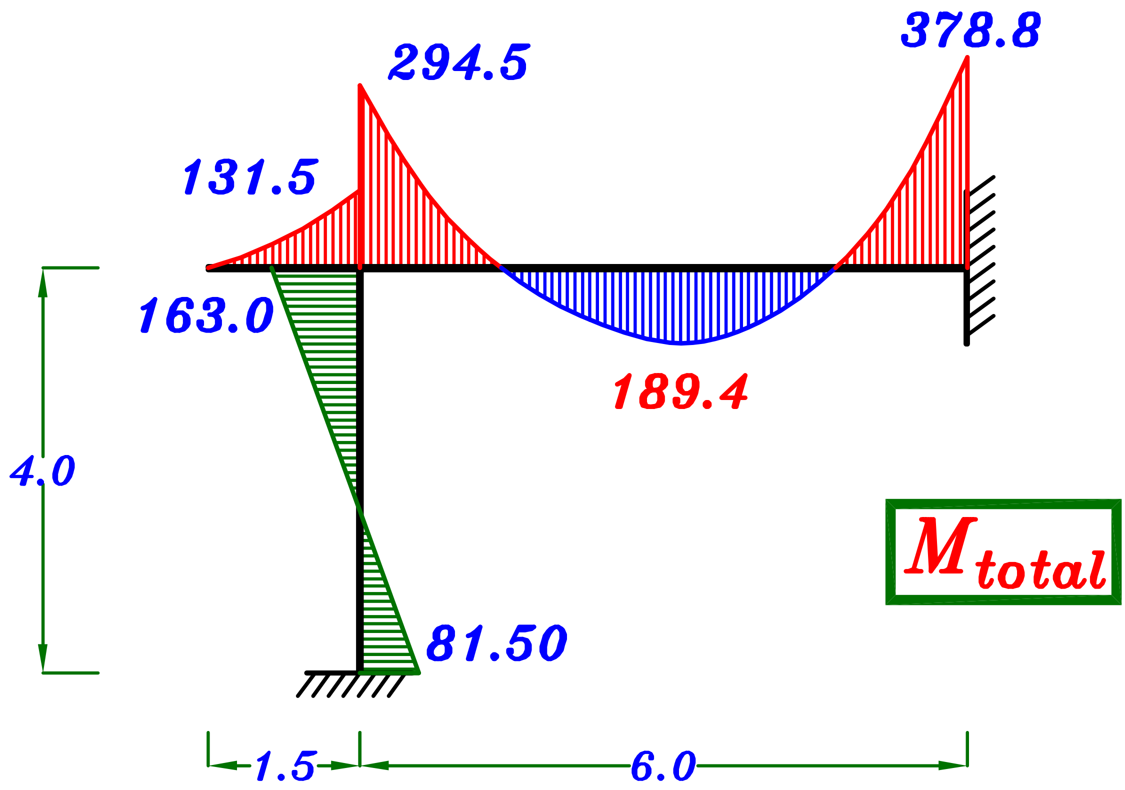
$$F.E.M.(ba) = +\frac{wL^2}{12} = +\frac{116.9 * 6.0^2}{12} = +350.7 \ kN.m.$$

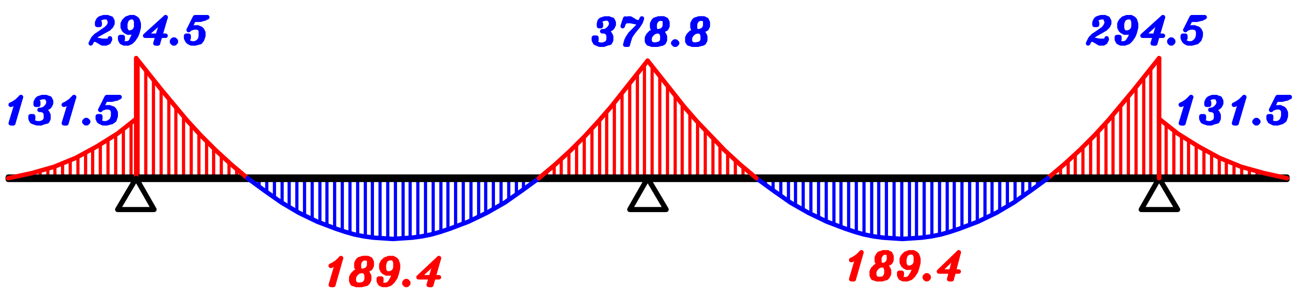
$$F.E.M.(ac) = +\frac{wL^2}{2} = +\frac{116.9 * 1.5^2}{2} = +131.5 \ kN.m.$$



Joint	c	a			b
member	c − a	a − c	a − d	a − b	b − a
D.F.	0	0.744	0	0.256	0
F.E.M.	0	0	+131.5	−350.7	+350.7
B.M.	0	+163.0	0	+56.2	0
C.O.M.	+81.5	0	0	0	+28.1
B.M.	0	0	0	0	0
M_F	+81.5	+163.0	+131.5	−294.5	+378.8

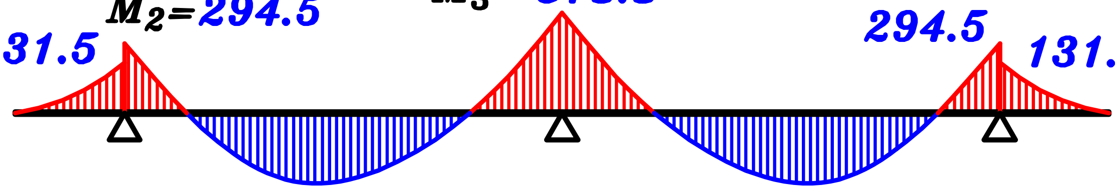

131.5
294.5
378.8
w= 116.9
d
a
w= 116.9
a
b
1.5
6.0
163.0
a
4.0
c
81.50


294.5
378.8
131.5
163.0
189.4
4.0
Mtotal
81.50
1.5
6.0


294.5
378.8
294.5
131.5
131.5
189.4
189.4

5-Distribute the moment of the Frame on Column Strip and Field Strip.

M_{total}

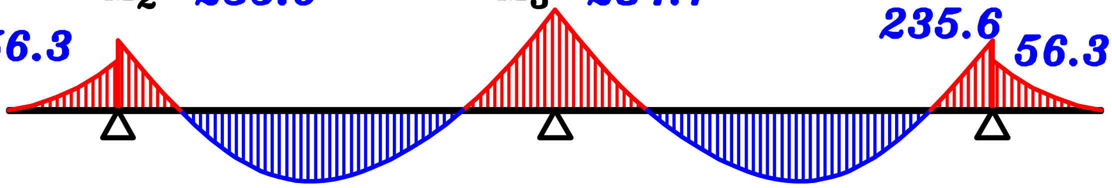


عرض ال C.S.

$M_1 * \frac{3.0}{7.0} = 56.3$

عرض الشريحه كله

moment C.S.

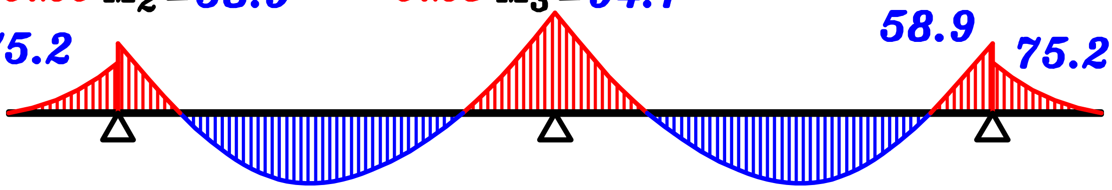


عرض ال F.S.

$M_1 * \frac{4.0}{7.0} = 75.2$

عرض الشريحه كله

moment F.S.

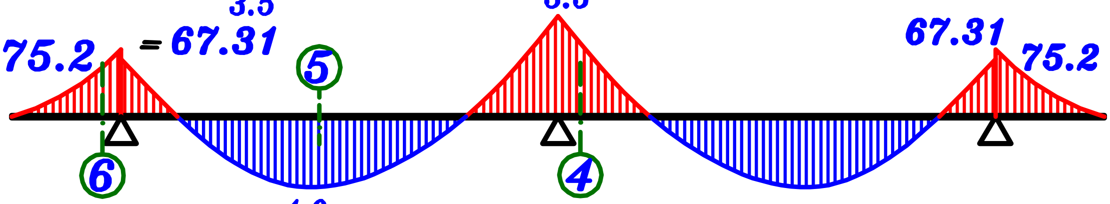


$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{4.0}{3.5}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.0}{3.5}}$$

moment F.S.

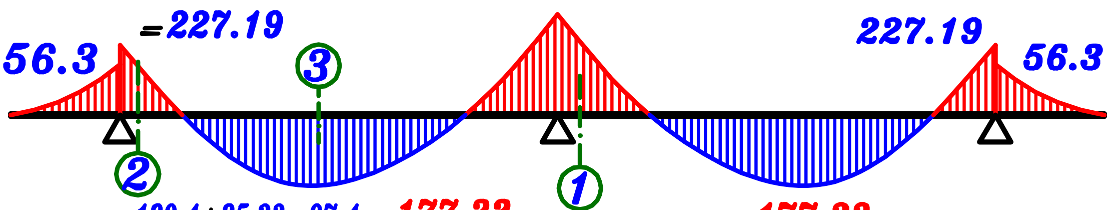
$b_{F.S.} = 4.0\ m$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

moment C.S.

$b_{C.S.} = 3.0\ m$

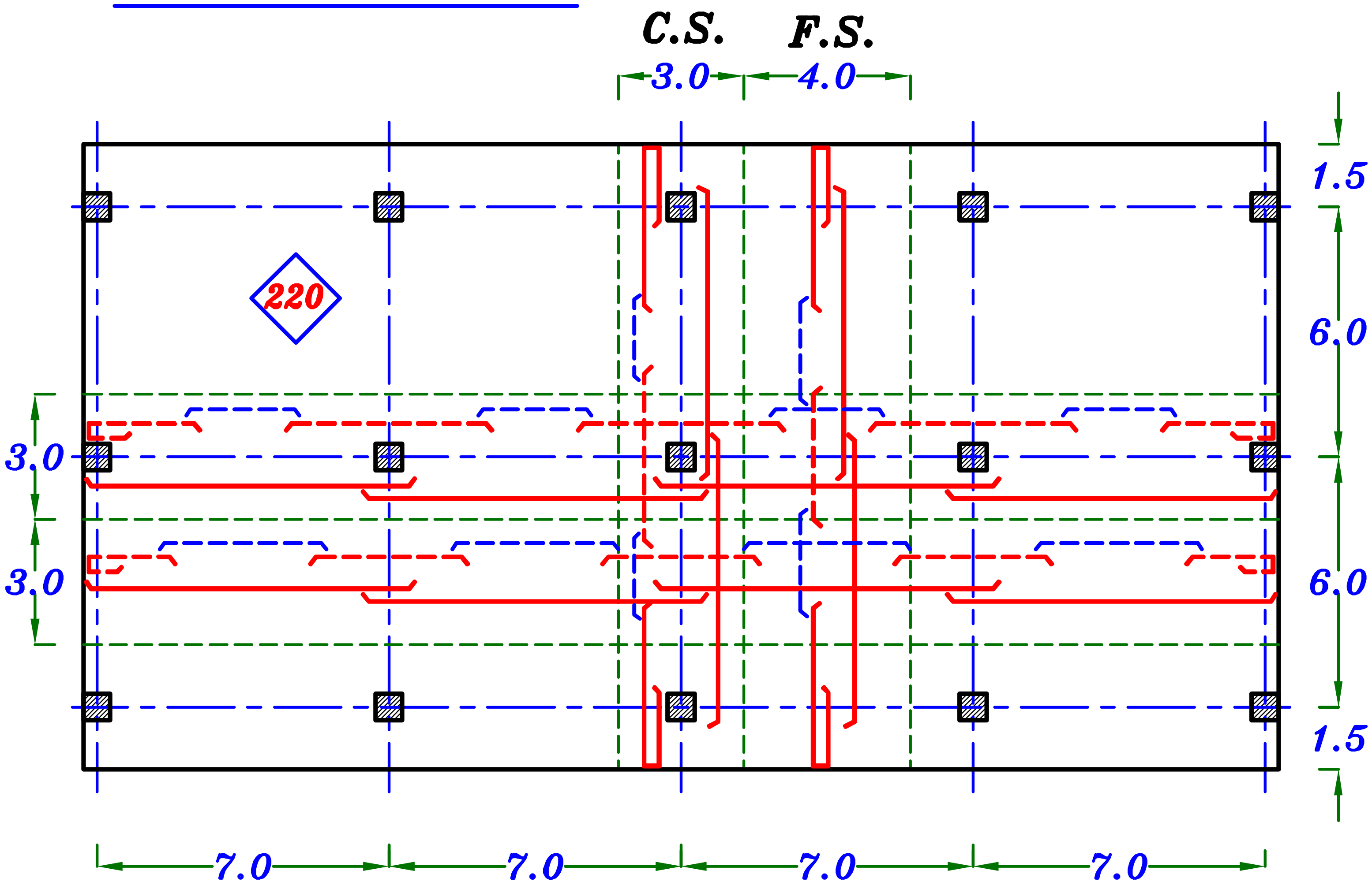


6-Design of sections. $d=t_s-40$ mm

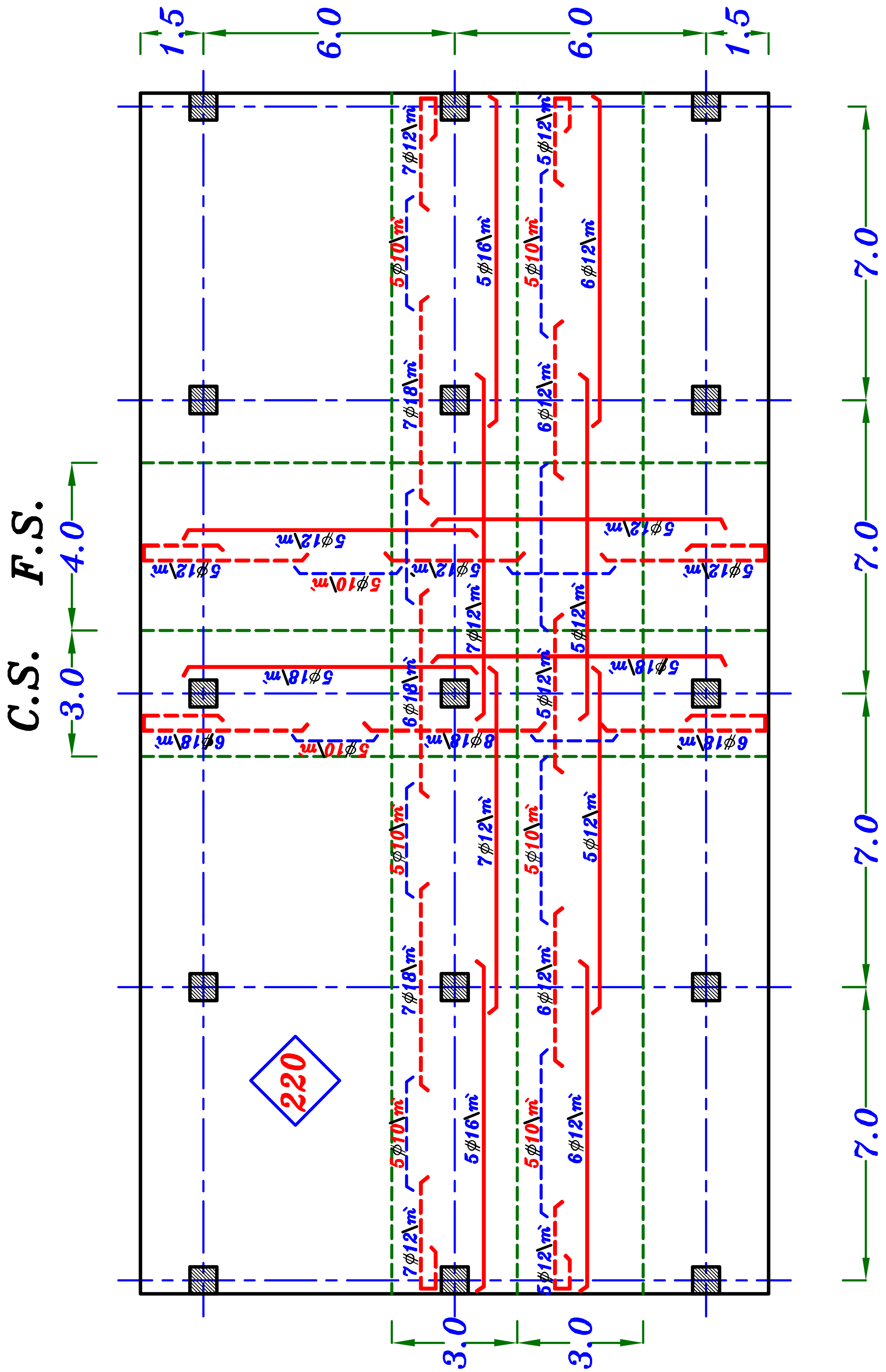
$d=t_s-40$ mm $=220-40=180$ mm

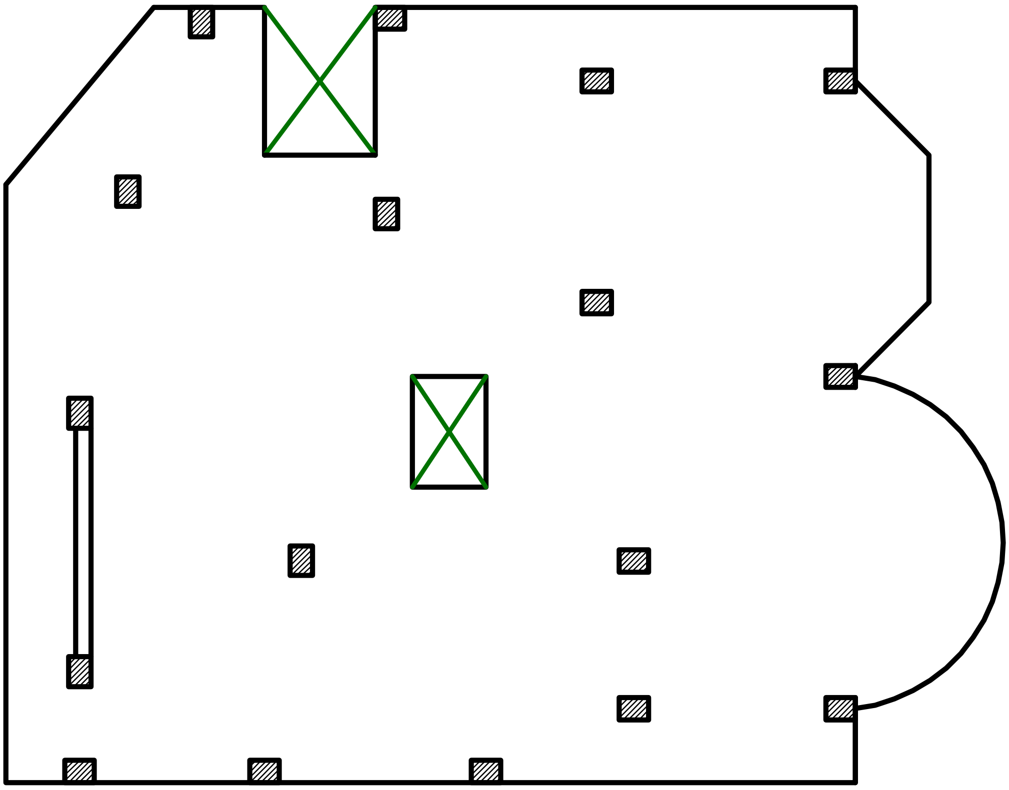
Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm²/b)	As (mm²/m)	No. of bars/m
Column Strip	1	270.6	3000	180	3.28	0.765	5458	1819	8ϕ18\m
	2	227.19	3000	180	3.58	0.785	4466	1488	6ϕ18\m
	3	177.23	3000	180	4.05	0.805	3397	1132	5ϕ18\m
Field Strip	4	108.2	4000	180	5.99	0.826	2021	505	5ϕ12\m
	5	97.4	4000	180	6.32	0.826	1819	454	5ϕ12\m
	6	75.2	4000	180	7.19	0.826	1404	351	5ϕ12\m

Details of RFT.

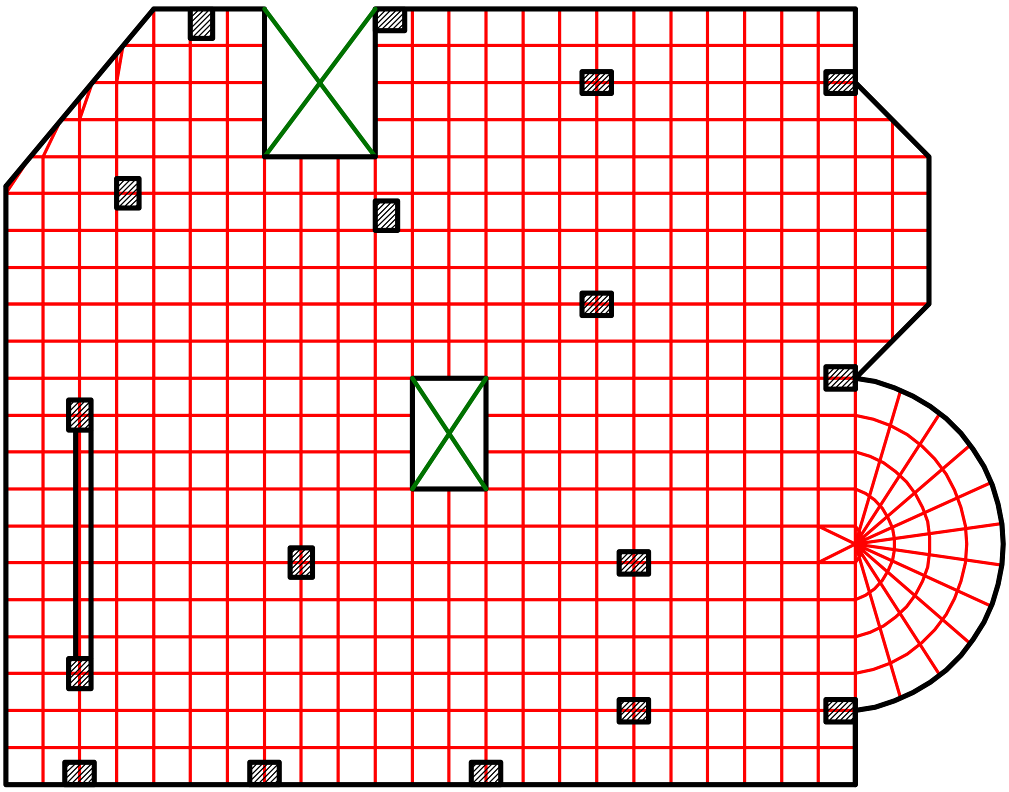


Details of RFT.





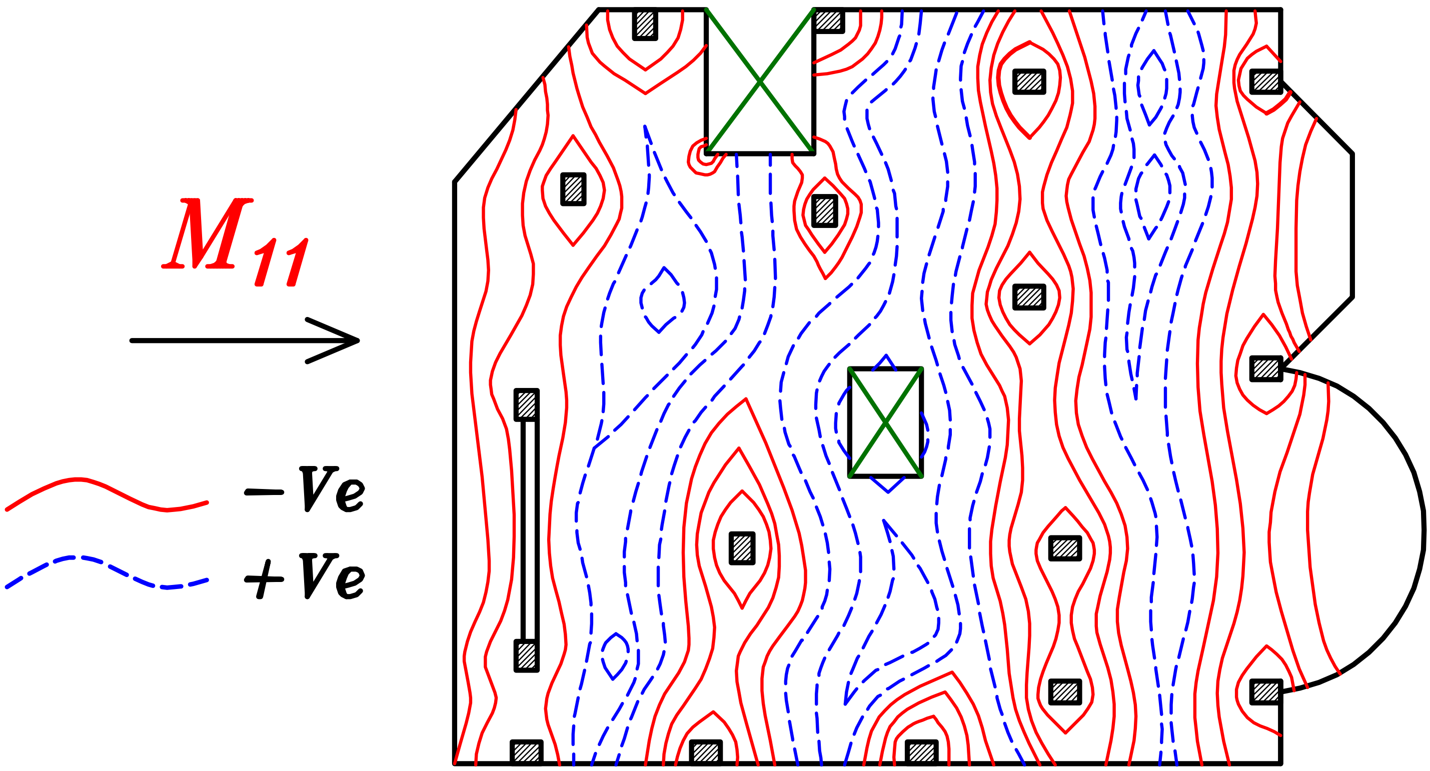
عند رسم البلاطه على الـ SAP يفضل تقسيم البلاطه الى Shells صغيره
و كلما صغرت ابعاد كل Shell كلما كانت الحسابات ادق .

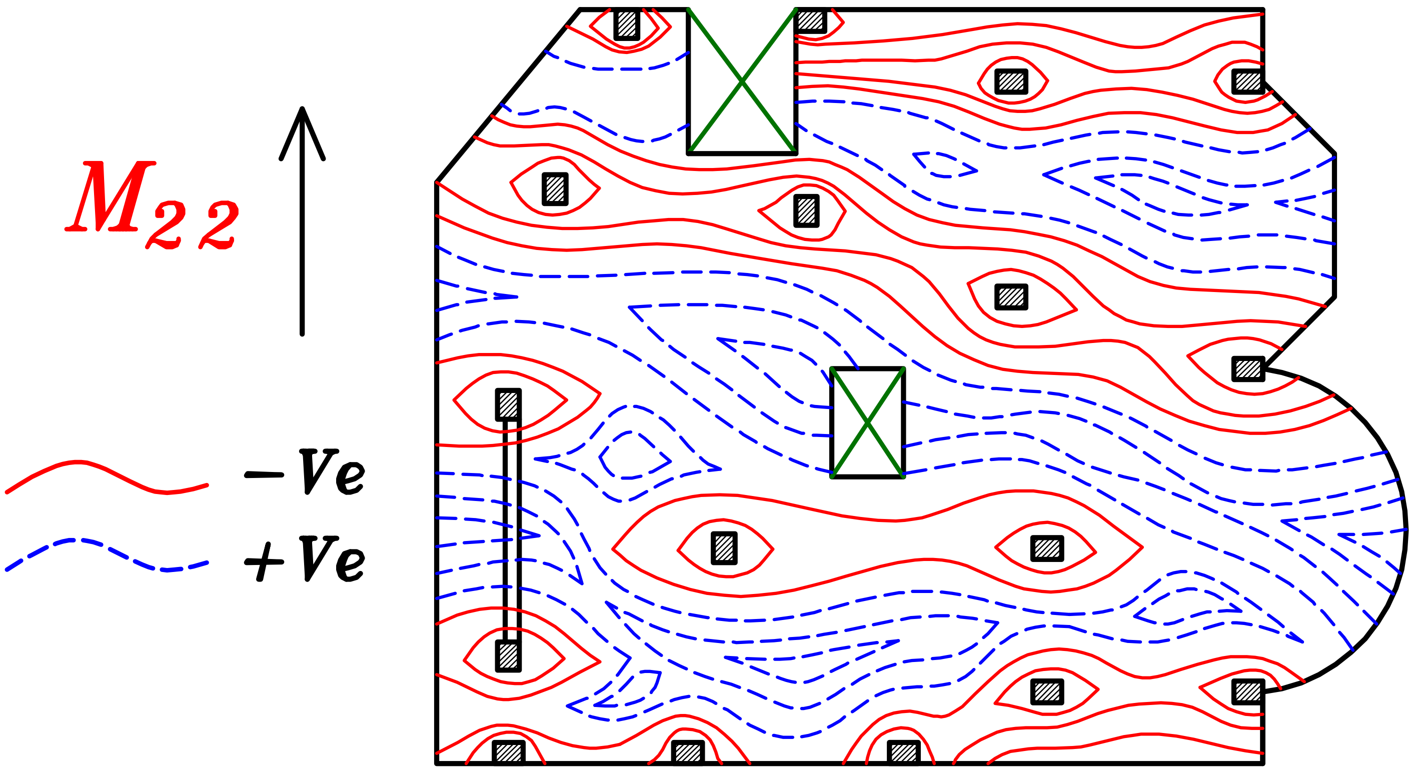


عند حل بلاطه **Flat Slab** عن طريق برنامج ال **SAP**

فان البرنامج يظهر الناتج لقيم العزوم على شكل خريطه كنتوريه لقيم العزوم فى اتجاهين .

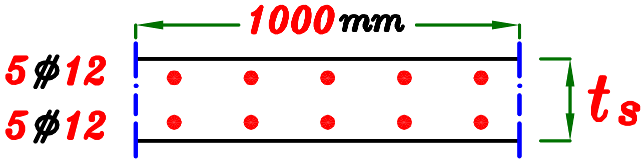
الاتجاه الاول فى اتجاه X و يسمى M_{11} و الاتجاه الثانى فى اتجاه Y و يسمى M_{22}





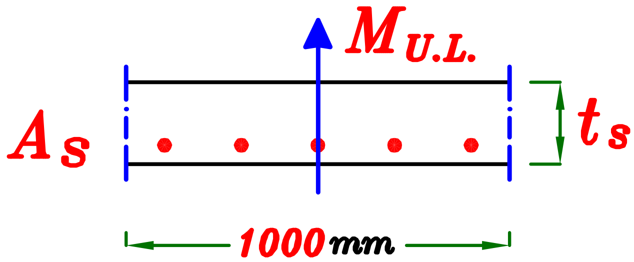
و لتسليح البلاطه نفرض اولا وضع شبكه تسليح سفليه $5\phi12\backslash m'$

و شبكه تسليح علويه $5\phi12\backslash m'$



فيكون قطاع الذى عرضه ١,٠ – متر فى البلاطه

فيتم حساب قيمه العزم الذى يتحمله هذا القطاع



$\boxed{A_s = 5\phi12}$

اى يتم حساب $M_{U.L.}$ للقطاع

و لان $A_s = A_s'$ للقطاع

اذا $M_{U.L.}(+ve) = M_{U.L.}(-ve)$

ممكن حساب قيمه $M_{U.L.}$ عن طريق First Principles

او عن طريق قيمه تقريبيه و هى فرض قيمه $\boxed{J \simeq 0.80}$

ثم تحديد قيمه $M_{U.L.}$ من قانون

$$A_s = \frac{M_{U.L.}}{J\ F_y\ d} = \frac{M_{U.L.}}{0.80\ F_y\ d} \xrightarrow{Get} M_{U.L.}$$

لتسليح اتجاه M_{11}

- شبكتين التسليح $5\phi12$ ستكون كافيه لاى قيم عزوم على خطوط الكنتور اقل من $M_{U.L.}$ سواء سفلى او علوى

- اى قيم على خطوط الكنتور اكبر من $M_{U.L.}$ ستحتاج لتسليح اضافى

و يتم تصميم كل قطاع منهم لمعرفه كميه التسليح الاضافى المطلوبه سواء كان حديد اضافى سفلى او حديد اضافى علوى .

و لتسليح اتجاه M_{22}

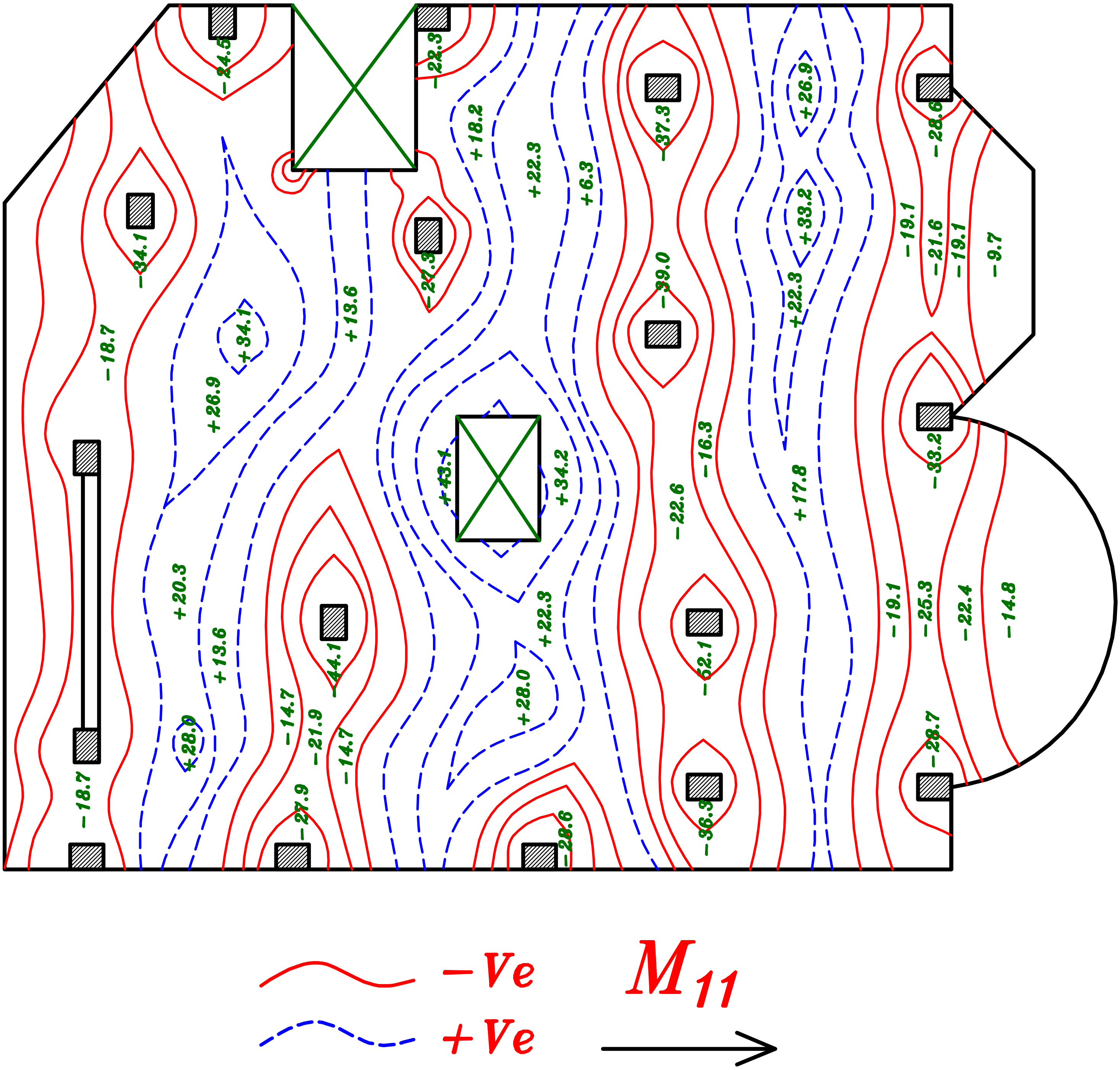
نكرر نفس الخطوات و لكن فى الاتجاه الاخر .

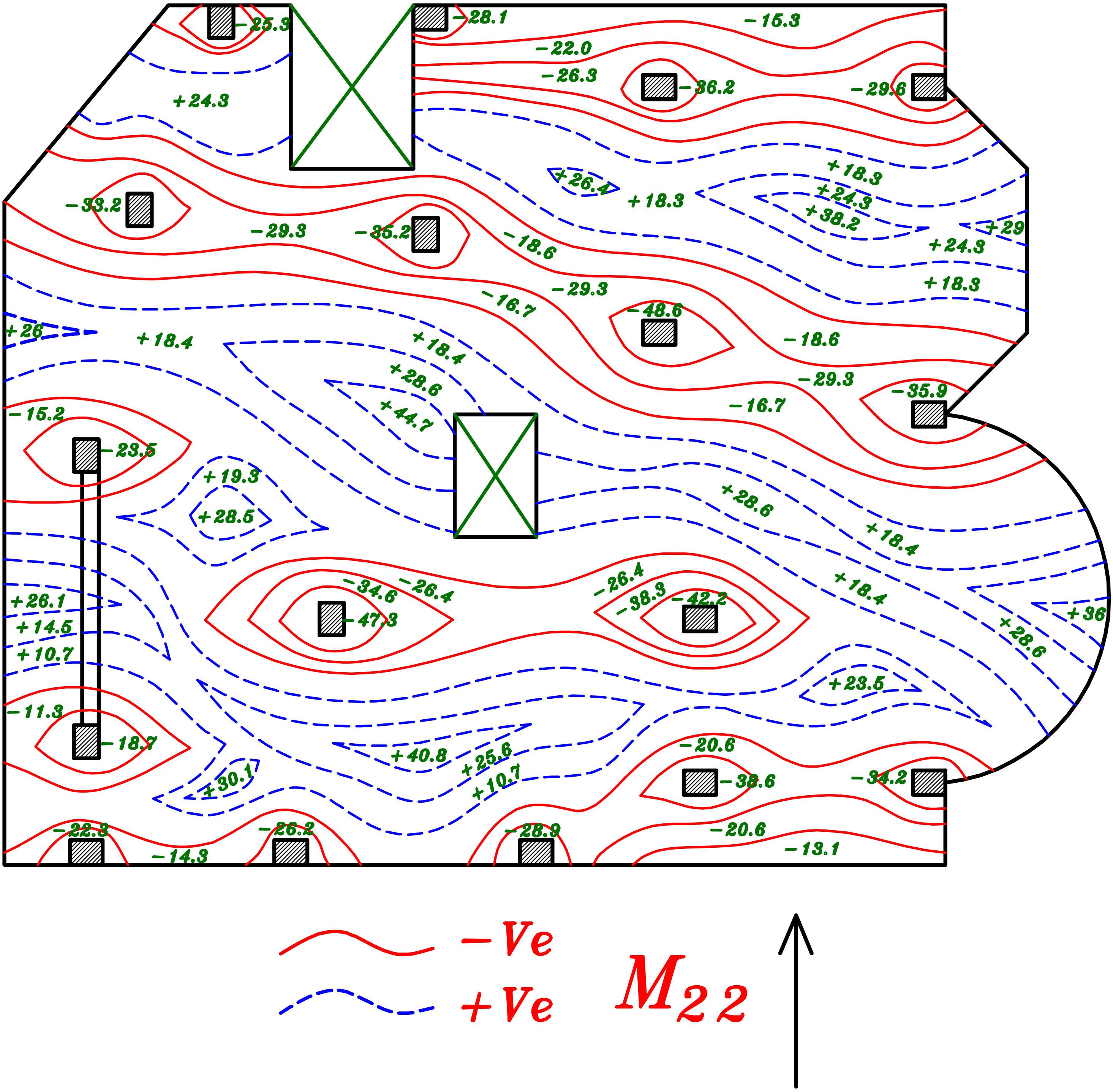
Example.

$t_s = 220$ mm , $F_{cu} = 25$ N\mm^2 , $F_y = 360$ N\mm^2

Design the given Plan according the SAP moment values. and draw details of Reinforcement in Plan.

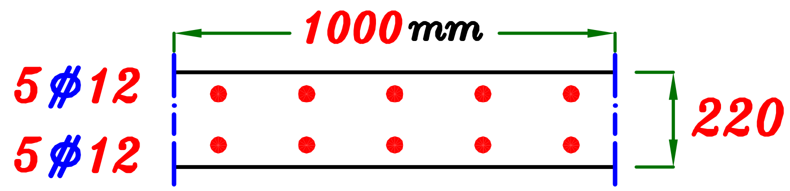
Moment Value are U.L. and kN.m





-25.3
-28.1
-15.3
-22.0
-26.3
-36.2
-29.6
+24.3
+18.3
+26.4
+18.3
+24.3
+38.2
+29
+24.3
-33.2
-29.3
-35.2
-18.6
+18.3
-16.7
-29.3
-48.6
-18.6
+26
+18.4
+18.4
+28.6
+44.7
-29.3
-35.9
-16.7
-15.2
-23.5
+19.3
+28.5
+28.6
+18.4
-34.6
-26.4
-47.3
-26.4
-38.3
-42.2
+18.4
+26.1
+14.5
+10.7
+36
+28.6
+23.5
-11.3
-18.7
+40.8
+25.6
+10.7
+30.1
-20.6
-38.6
-34.2
-22.3
-26.2
-28.9
-20.6
-14.3
-13.1
-Ve
+Ve
M_{22}

نضع شبكه تسليح سفليه و علويه $5\phi12\backslash m'$



$$d = t_s - 30\ mm = 220 - 30 = 190\ mm$$

$$A_s = 5\phi12 = 565\ mm^2$$

Assume $J \simeq 0.80$

$$\because A_s = \frac{M_{U.L.}}{J\ F_y\ d}$$

$$\therefore 565 = \frac{M_{U.L.}}{0.80 * 360 * 190} \longrightarrow M_{U.L.} = 30916800\ N.mm$$

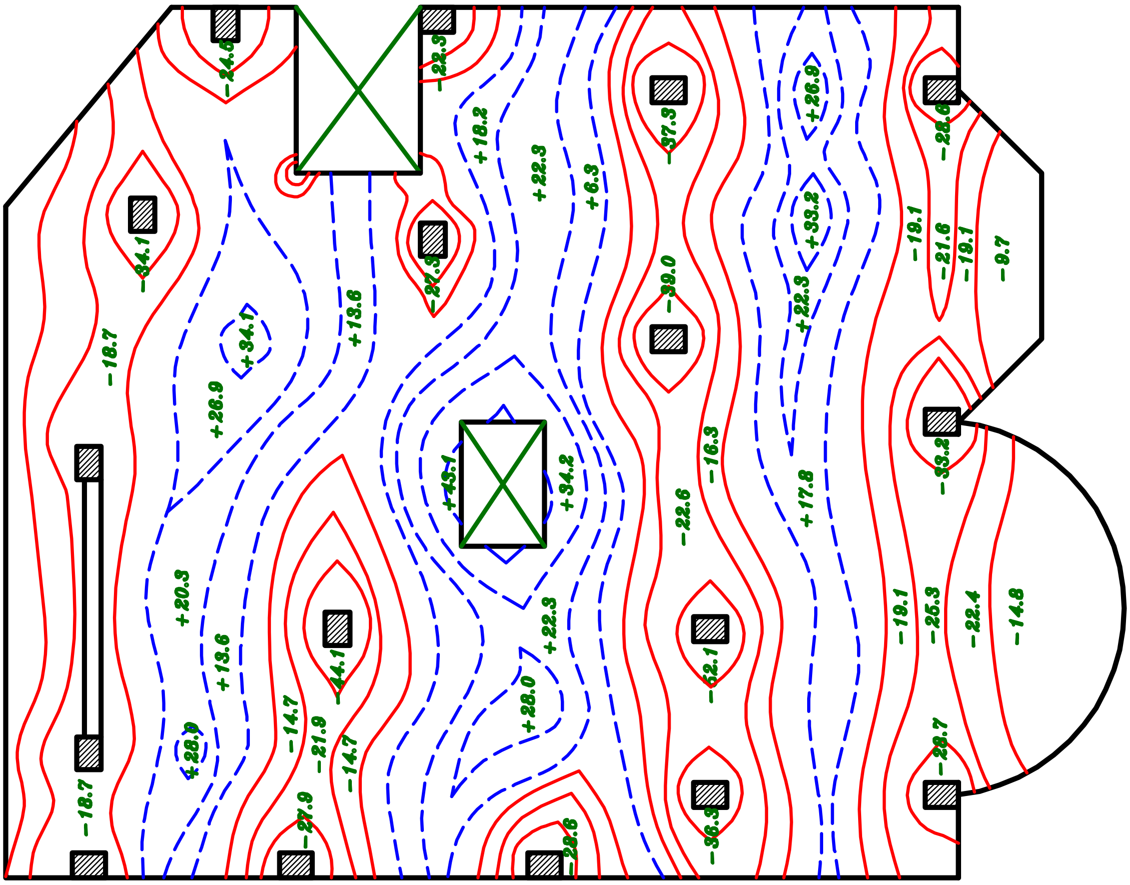
$$\therefore \boxed{M_{U.L.} = 30.91\ \ kN.m}$$

الشبكه $5\phi12\backslash m'$ ستتحمل حتى عزم $30.91\ kN.m$

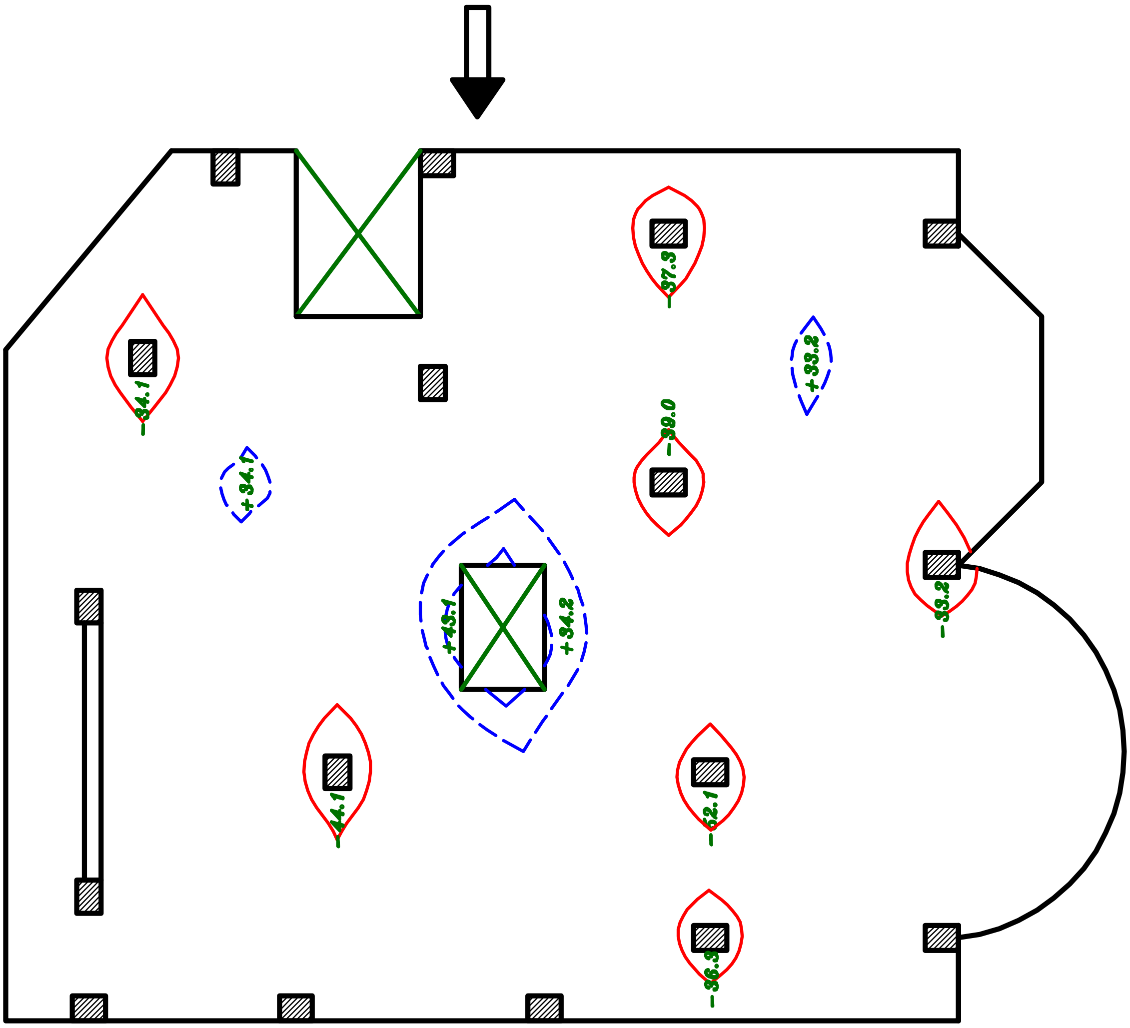
اذا المناطق التى يوجد بها العزوم اقل من $30.91\ kN.m$
لن نحتاج لوضع حديد اضافى بها

المناطق التى بها عزم سفلى او علوى اكبر من $30.91\ kN.m$
سيحتاج لحديد اضافى .

RFT. For M_{11}

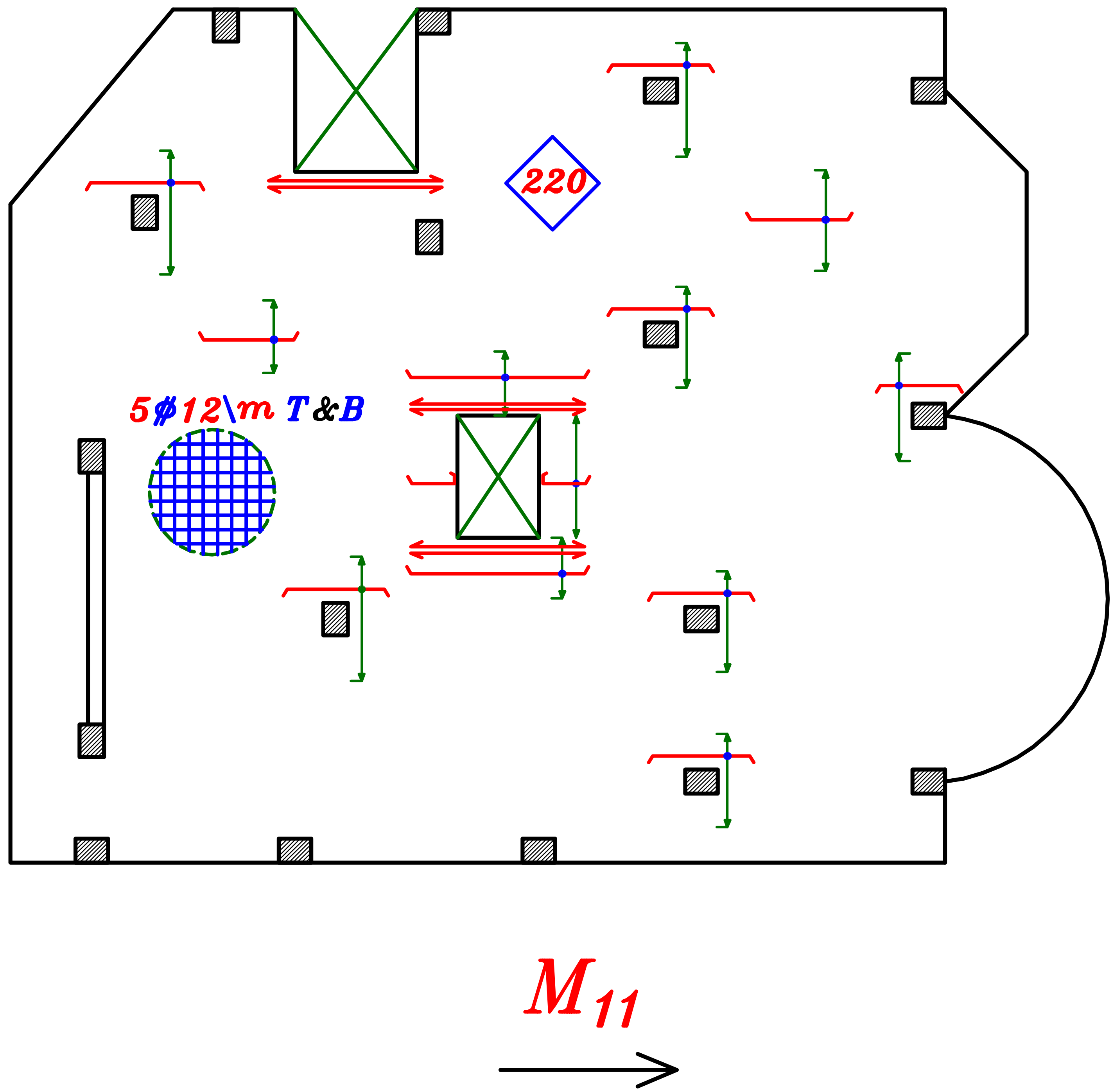


-24.6
-22.3
+18.2
+22.3
+6.3
-37.3
+26.9
-28.6
-34.1
-37.3
+13.6
-39.0
+33.2
+22.3
-19.1
-21.6
-19.1
-9.7
+34.1
-18.7
+26.9
-16.3
-33.2
+43.1
+34.2
-22.6
+17.8
+20.3
+22.3
-19.1
-25.3
-22.4
-14.8
+13.6
-44.1
-52.1
+28.0
-14.7
-21.9
+28.0
-14.7
-28.7
-18.7
-27.9
-28.6
-36.3

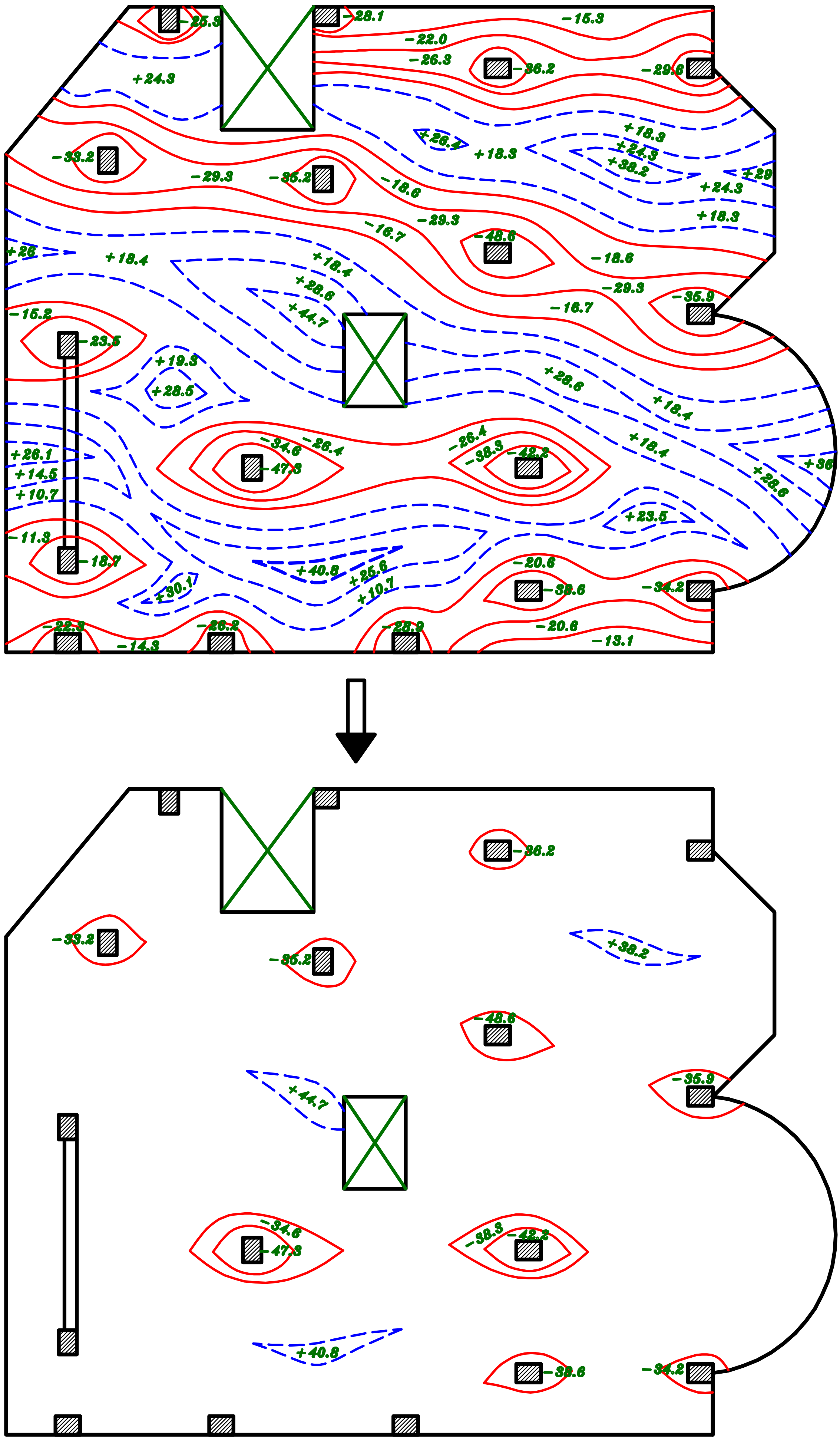


-37.3
-34.1
+33.2
-39.0
+34.1
-33.2
+43.1
+34.2
-44.1
-52.1
-36.3

يتم وضع حديد اضافى فى المناطق التى العزوم سفلاً او علوى اكبر من **30.91 kN.m**

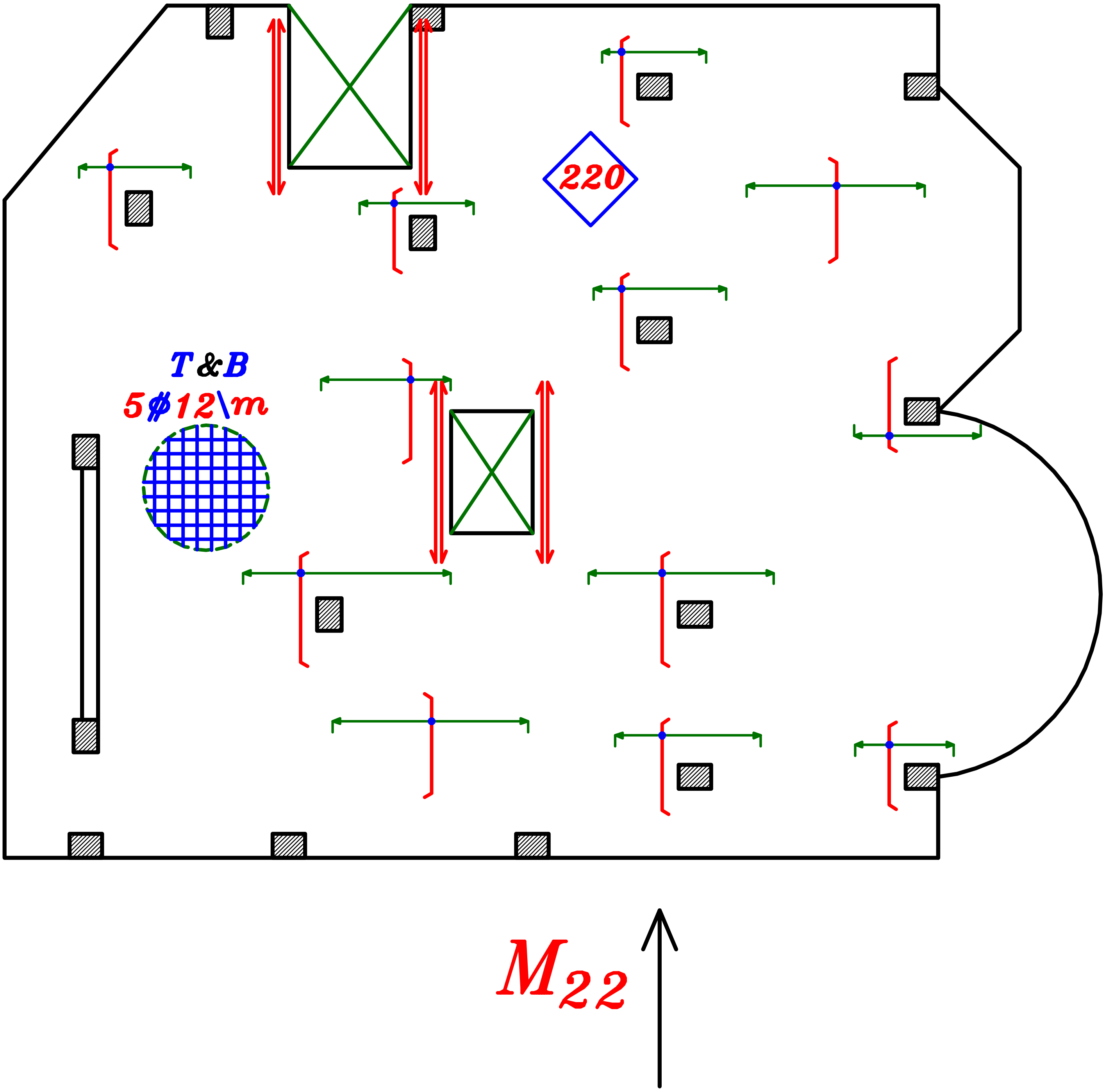


RFT. For M_{22}

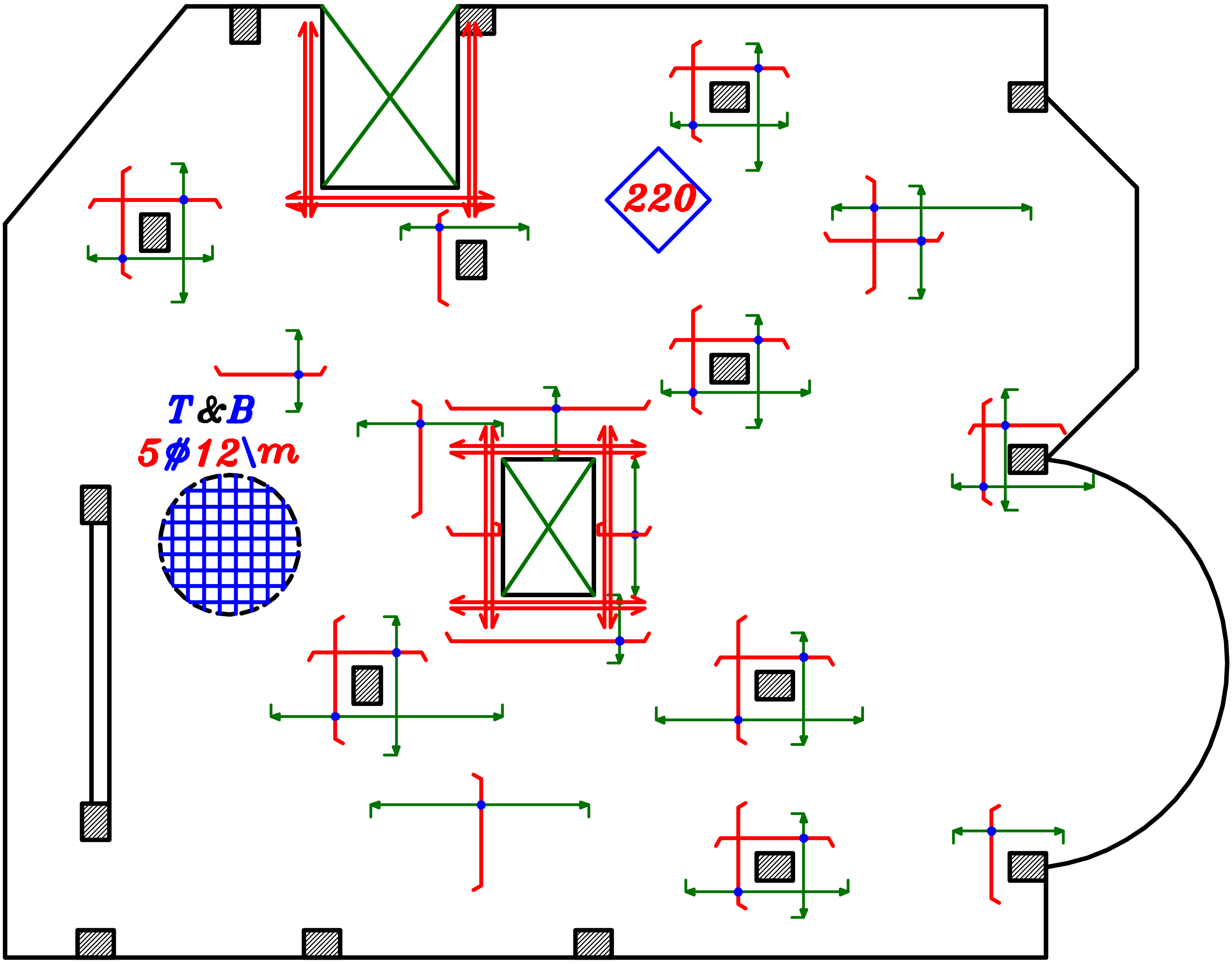


−25.9
−28.1
−15.3
−22.0
−26.3
−36.2
−29.6
+24.3
+26.7
+18.3
+18.3
+24.3
+38.2
+29
+24.3
+18.3
−33.2
−29.3
−35.2
−18.6
−29.3
−16.7
−48.6
−18.6
+26
+18.4
+18.4
+28.6
+44.7
−29.3
−35.9
−16.7
−15.2
−23.5
+19.3
+28.5
+28.6
+18.4
+18.4
−34.6
−26.4
−47.3
−26.4
−38.3
−42.2
+36
+26.1
+14.5
+10.7
+28.6
+23.5
−11.3
−18.7
+40.8
+25.6
+10.7
+30.1
−20.6
−38.6
−34.2
−22.3
−14.3
−26.2
−28.9
−20.6
−13.1
−36.2
−33.2
−35.2
+38.2
−48.6
−35.9
+44.7
−34.6
−47.3
−38.3
−42.2
+40.8
−38.6
−34.2

يتم وضع حديد اضافى فى المناطق التى العزوم سفلاً او علوى اكبر من **30.91 kN.m**



Total RFT. of the Flat Slab.
Main Meshes & Additional RFT.



M_{11} & M_{22}

Flat Slab General Examples.

Steps of Design.

· Flat Slabs خطوات تصميم ال

1– Get concrete Dimensions For the slab elements.

a – Columns Dimensions. ($b_{col.}$)

b – Slab thickness (t_s) .

c – Drop Panel Dimensions. ان وجدت

2– Calculate the loads on the slab (w_s) .

3– Check punching.

4– Take a Strips in the slabs at the long and short directions.
The strip width From C.L. the slab to C.L. the slab.

& Draw B.M.D. For the Strip (M_o) Using

a– Empirical Method.

b – Frame Analysis Method.

5– Distribute the moment on both
Column Strip & Field Strip

6– Design the sections of the slab using C_1 & J

7– Draw Details of RFT. of the slab in plan.

Example.

The given plan shows general layout of a part plan of 5 storey building.

Data. $F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

F.C.+ walls $= 3.0\ kN\backslash m^2$, L.L. $= 6.0\ kN\backslash m^2$

The Floor height = 4.0 m

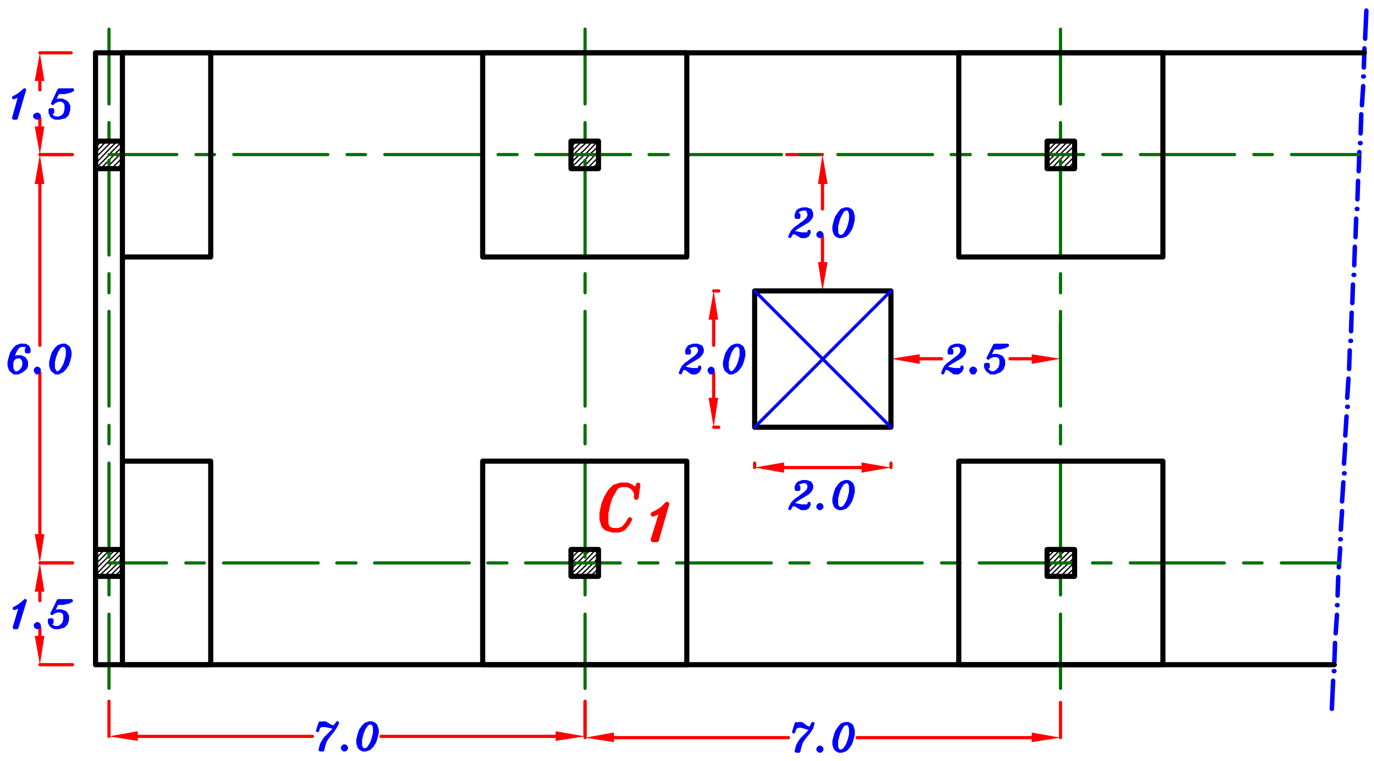
Req.

① Check punching on column C_1

② Complete design of typical Floor in both directions.

③ Design column C_1 at Ground Floor.

④ Draw details of reinforcement in plan.



Part Plan

Solution.

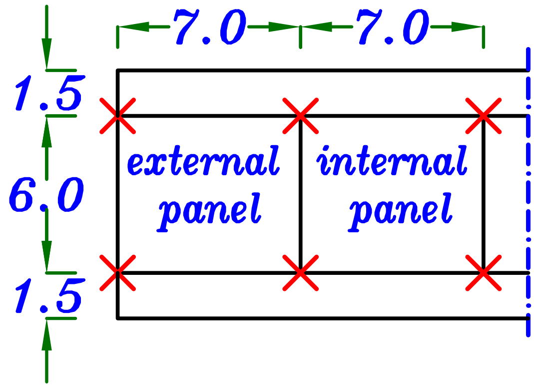
1- Concrete Dimensions.

Column dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{4000}{15} = 266.6\ mm \\ \dfrac{L_1}{20} = \dfrac{7000}{20} = 350\ mm \end{cases}$$

$$\boxed{b_{Col.} = 350\ mm}$$

$(350 * 350)$



Slab Thikness.

$L_1 = 7.0\ m$

External panel $t_s = \dfrac{L_1}{36} = \dfrac{7000}{36} = 194.4\ mm$

Internal panel $t_s = \dfrac{L_1}{40} = \dfrac{7000}{40} = 175\ mm$

Cantilever $t_s = \dfrac{L_c}{10} = \dfrac{1500}{10} = 150\ mm$

$$\boxed{t_s = 200\ mm}$$

Drop Panel.

Take $t_d = \dfrac{t_s}{2} = 100\ mm$

Take Width of drop panel $X = \dfrac{L_2}{2} = 3.0\ m$ فى الإتجاهين

2- Loads on the Slab.

$$w_{s\,U.L.} = 1.4\left[\left(t_s + \frac{t_d}{4}\right)\gamma_c + F.C. + Wall\right] + 1.6\,(L.L.)$$

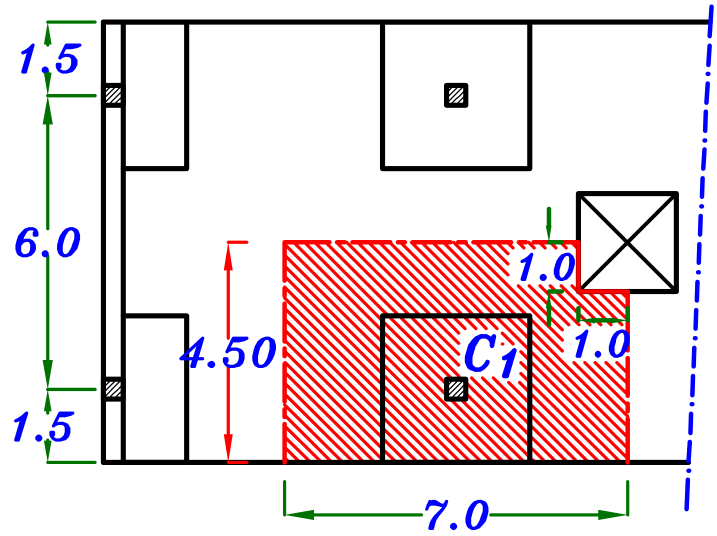
$$w_{s\,U.L.} = 1.4\left[\left(0.20 + \frac{0.1}{4}\right) * 25 + 3.0\right] + 1.6\,(6.0) = 21.68\ kN\backslash m^2$$

3- Check Punching on interior column

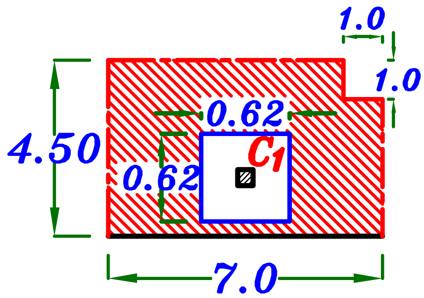
كل عمود يحمل مساحه

من $C.L.$ البلاطه

الى $C.L.$ البلاطه الاخرى



$\underline{\underline{C_1}}$ **Interior Column.**



$$d = t_s + t_d - 30\,mm = 200 + 100 - 30 = 270\ mm = 0.27\,m$$

$$C + d = 0.35 + 0.27 = 0.62\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 21.68\left[7.0 * 4.5 - 1.0 * 1.0 - 0.62 * 0.62\right] = 653.0\ kN$$

$$A_p = (b_o * d) = (4 * 620) * 270 = 669600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{653.0 * 10^3}{669600} * 1.15 = 1.12\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\delta_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

$$\boxed{q_{pu} < q_{pcu}} \longrightarrow \text{Safe punching}$$

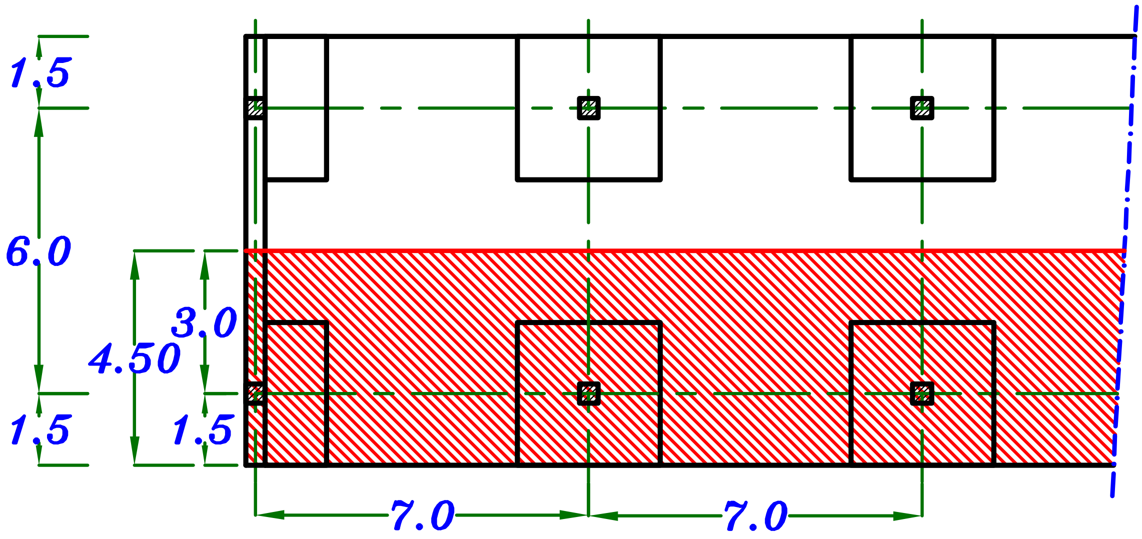
Moment at Long Direction.

Use Empirical Method.

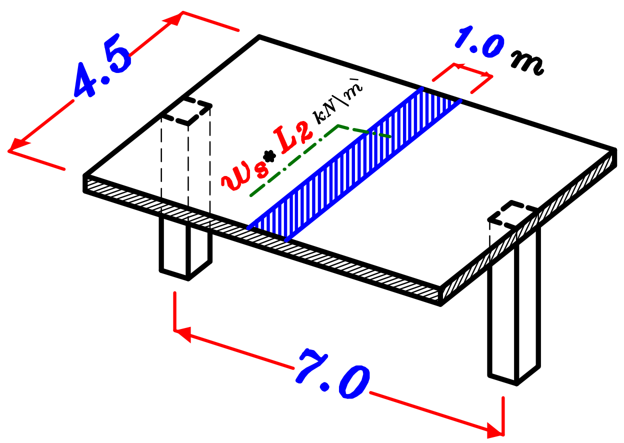
فى الاتجاه الطويل نختار حساب العزوم بطريقه **Empirical Method** لانها أسهل .
و خاصه أن عدد البواكى فى هذا الاتجاه أكثر من ٣ بواكى .

يتم أخذ شريحه التصميم الكليه من C.L. البلاطه الى نهايه ال Cantilever

فيكون عرض شريحه التصميم الكليه $Total\ Strip\ width = \frac{6.0}{2.0} + 1.5 = 4.5\ m$



Total moment on the panel.



$Span = 7.0\ m$

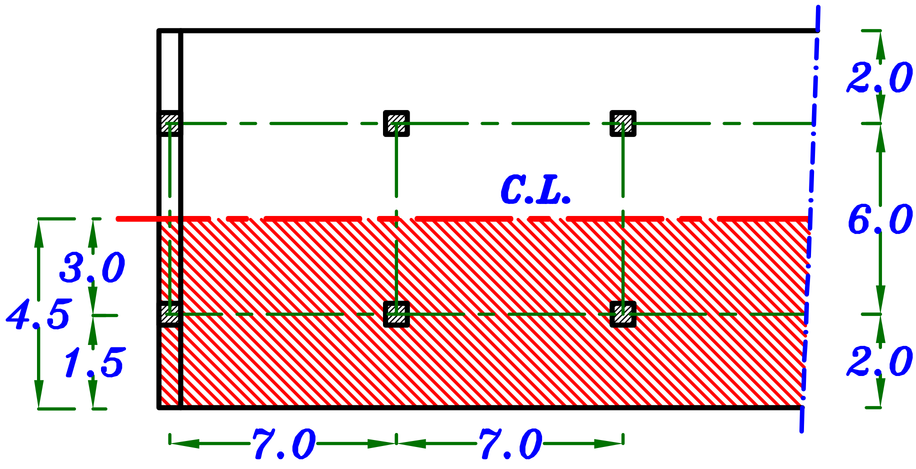
$Width = 4.50\ m$

Moment in Long Direction.

$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8} = \frac{(21.68 * 4.5)\left(7.0 - \frac{2}{3} * 0.35\right)^2}{8}$$

$$\boxed{M_o = 558.38\ kN.m}$$ Long Direction

Modification Factor.



عرض شريحه التصميم الكليه

Total Strip width =

$$= \frac{6.0}{2.0} + 1.5 = 4.5\ m$$

$$\boxed{b_{C.S.} = \frac{L_2}{4} + \text{Width of the Cantilever}}$$

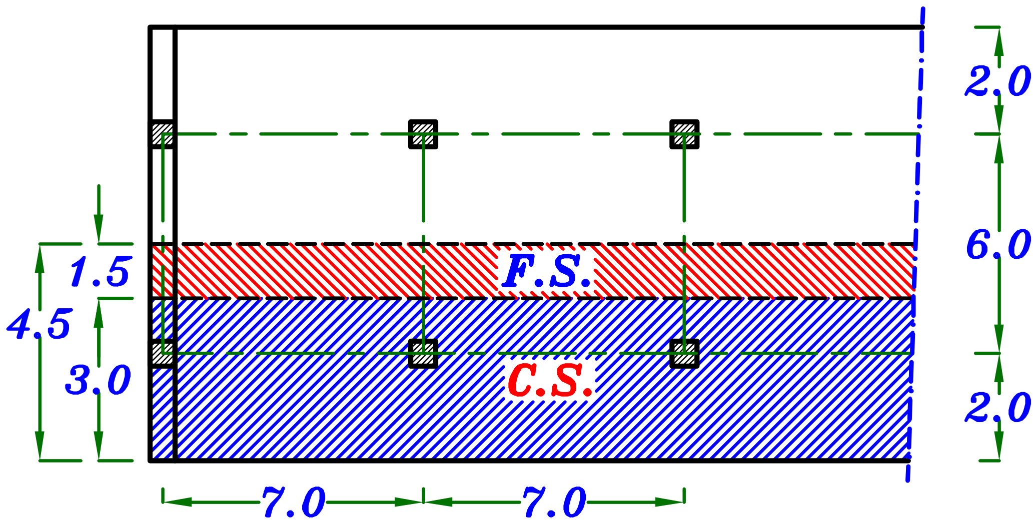
يؤخذ عرض ال **Column strip**

$$b_{C.S.} = \frac{6.0}{4} + 1.5 = 3.0\ m$$

$$\boxed{b_{F.S.} = \text{Total Strip width} - b_{C.S.}}$$

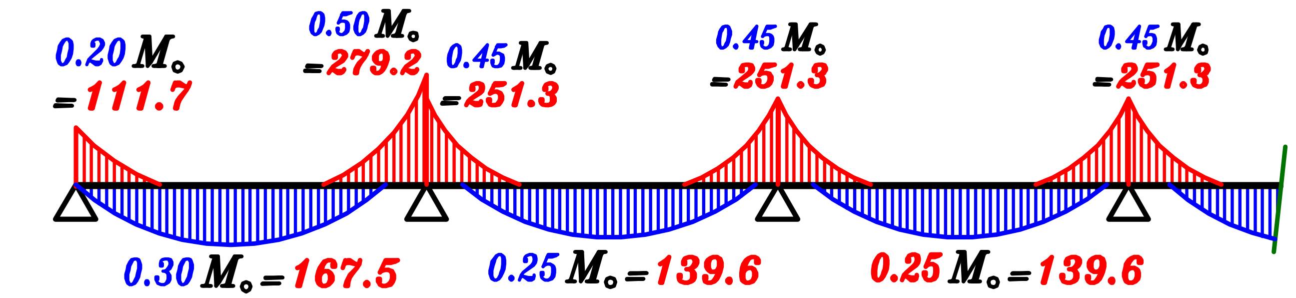
و يؤخذ عرض ال **Field strip**

$$b_{F.S.} = 4.50 + 3.0 = 1.50\ m$$



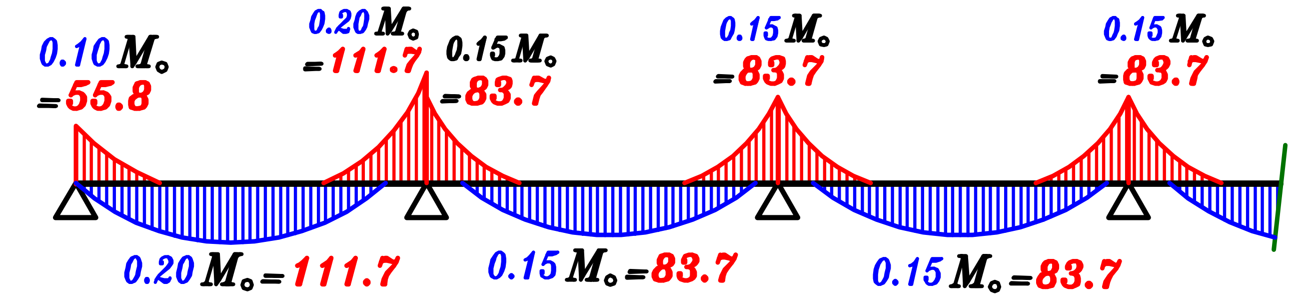
Modification Factor For Field Strip

$$\boxed{M.F. = \frac{\text{Field strip العرض الحقيقى لل}}{\text{عرض الشريحه الكلى } \frac{١}{٢}} = \frac{1.5}{2.25} = 0.67}$$



C.S.

$(M_{C.S.})$

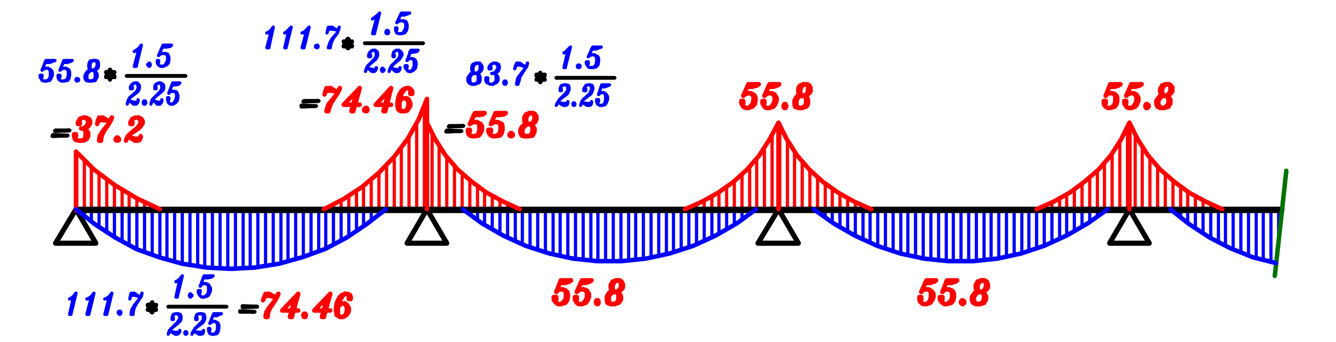


F.S.

$(M_{F.S.})$

$$\text{Modification Factor} = \frac{1.5}{2.25} = 0.67$$

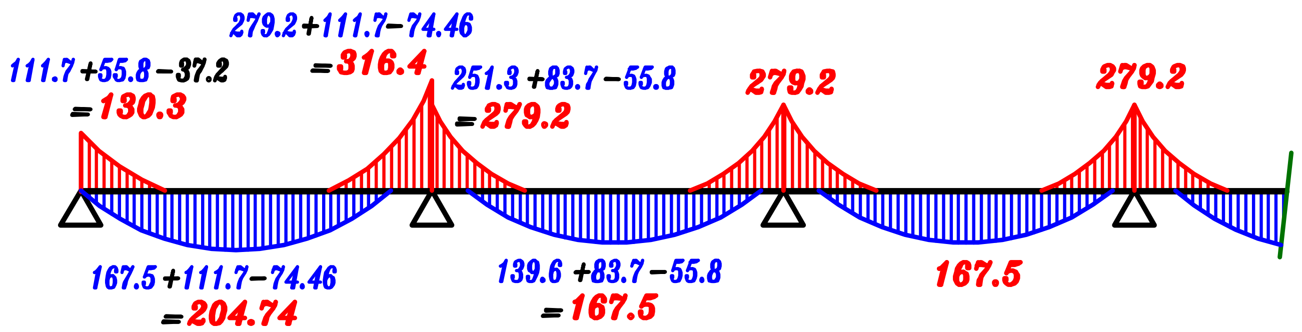
$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{1.5}{2.25}}$$



F.S.

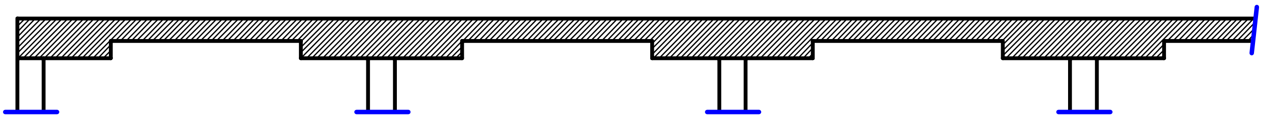
$(M_{F.S.})_{mod.}$

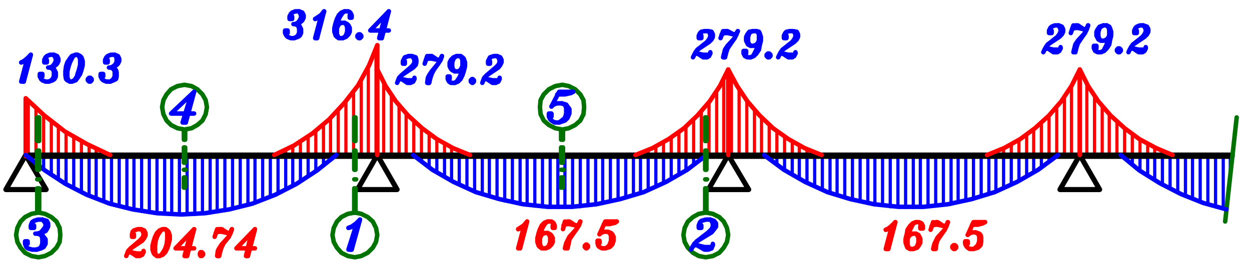
$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$



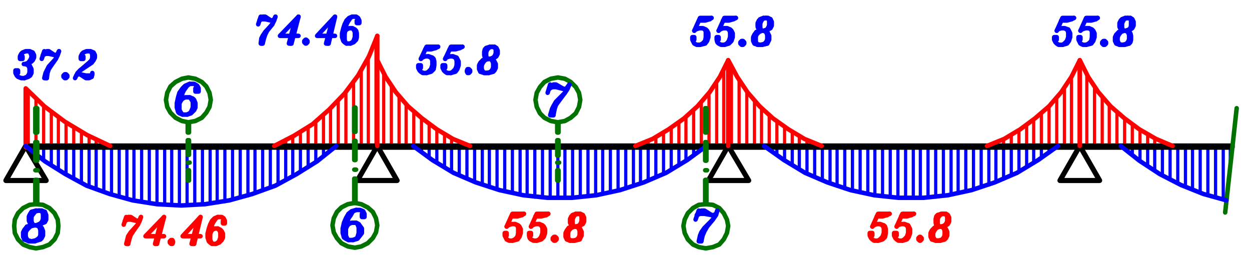
C.S.

$(M_{C.S.})_{mod.}$





C.S.
($M_{C.S.}$) mod.



F.S.
($M_{F.S.}$) mod.

Design of sections.

$d = t_s - 30$ mm

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m'
Column Strip	1	316.4	3000	270	4.15	0.808	4028	1342	7ϕ16\m
	2	279.2	3000	270	4.42	0.817	3515	1171	6ϕ16\m
	3	130.3	3000	270	6.47	0.826	1622	540	5ϕ12\m
	4	204.74	3000	170	3.25	0.765	4373	1457	8ϕ16\m
	5	167.5	3000	170	3.59	0.786	3482	1160	6ϕ16\m
Field Strip	6	74.46	1500	170	3.81	0.795	1530	1020	6ϕ16\m
	7	55.8	1500	170	4.40	0.815	1118	745	7ϕ12\m
	8	37.2	1500	170	5.39	0.826	736	490	5ϕ12\m

Moment at Short Direction.

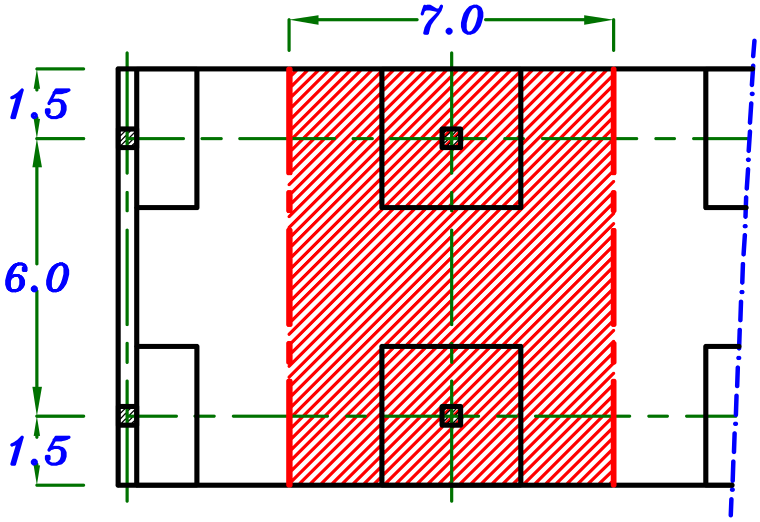
فى الاتجاه القصير نختار حساب العزوم بطريقه ***Frame analysis method***
لانه لن ينفع استخدام طريقه الـ ***Empirical*** لان عدد البواكى أقل من ٣ بواكى .

$$Span = L_2 = 6.0\ m$$

$$Width = L_1 = 7.0\ m$$

$$b_{C.S.} = \frac{L_2}{2} = 3.0\ m$$

$$b_{F.S.} = L_1 - \frac{L_2}{2} = 4.0\ m$$

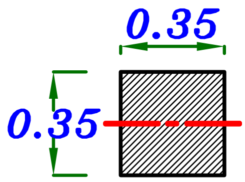


$$w = w_s * L_2 = 21.68 * 7.0 = 151.76\ kN/m$$

@ **Calculate Moment of Inertia For Slabs & Columns.**

عمود خارجى

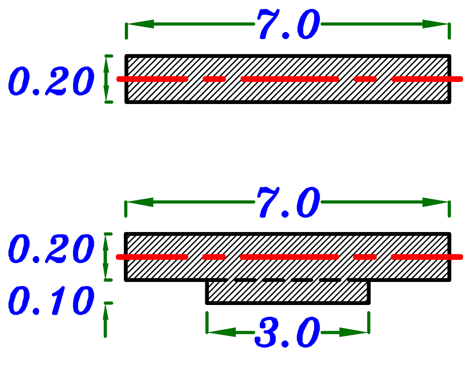
$$I_c = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.35 * 0.35^3}{12} = 7.50 * 10^{-4}\ m^4$$



$$I_{s1} = \frac{L_2 * t_s^3}{12} = \frac{7.0 * 0.20^3}{12} = 4.67 * 10^{-3}\ m^4$$

$$t_{av} = t_s + \frac{t_d}{4} = 0.20 + \frac{0.1}{4} = 0.225\ m$$

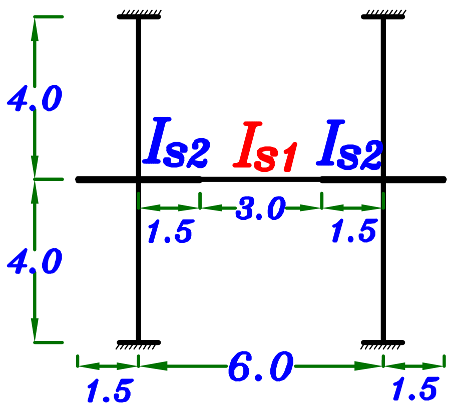
$$I_{s2} = \frac{L_2 * t_{av}^3}{12} = \frac{7.0 * 0.225^3}{12} = 6.64 * 10^{-3}\ m^4$$

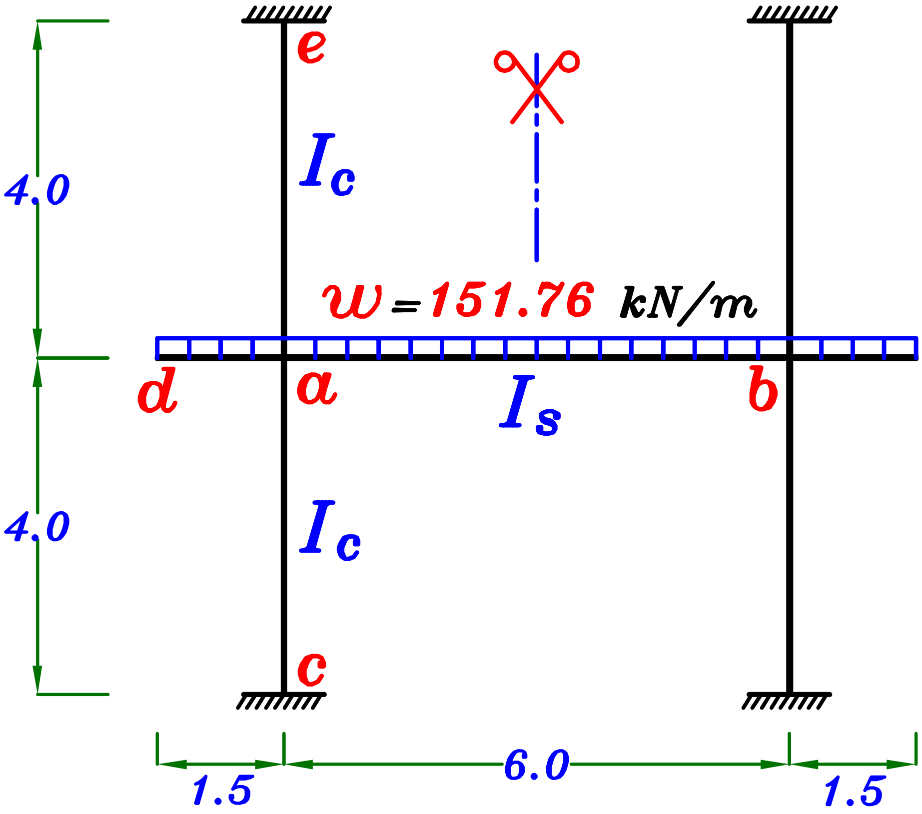


$$I_{s\,av} = \frac{I_{s1}(L_2 - X) + I_{s2}(X)}{L_2}$$

$$I_{s\,av} = \frac{(4.67 * 10^{-3})(3.0) + (6.64 * 10^{-3})(3.0)}{6.0}$$

$$I_{s\,av} = 5.65 * 10^{-3}\ m^4$$





ⓑ **Calculate the stiffnesss For each member.**

For Slabs.

$$K_{ab} = \frac{1}{2} * \frac{I_{sav}}{L} = \frac{1}{2} * \frac{5.65 * 10^{-3}}{6.0} = 4.71 * 10^{-4}$$

For Columns.

$$K_{ac} = K_{ae} = \frac{I_c}{h} = \frac{7.50 * 10^{-4}}{4.0} = 1.87 * 10^{-4}$$

For Cantilever

$$K_{ad} = Zero$$

ⓒ **Calculate the Distribution Factors. (D.F.)**

For Joint a

$$\Sigma K = K_S + 2K_C = 4.71 * 10^{-4} + 2 * 1.87 * 10^{-4} = 8.45 * 10^{-4}$$

$$D.F.(ac) = D.F.(ae) = \frac{1.87 * 10^{-4}}{8.45 * 10^{-4}} = 0.221$$

$$D.F.(ab) = \frac{4.71 * 10^{-4}}{8.45 * 10^{-4}} = 0.558$$

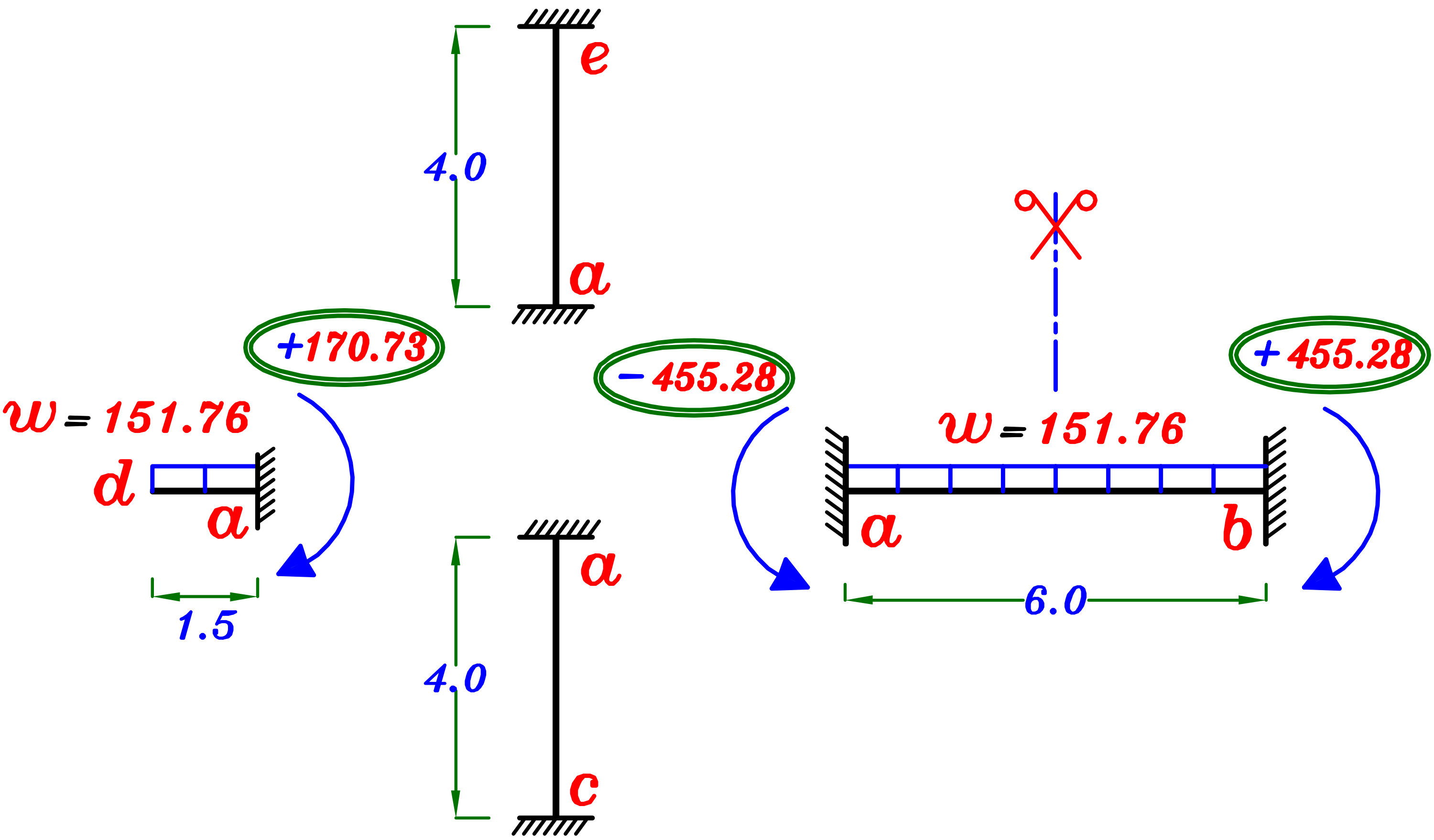
$$D.F.(ad) = Zero$$

ⓓ Calculate Fixed End Moment For the Slab.

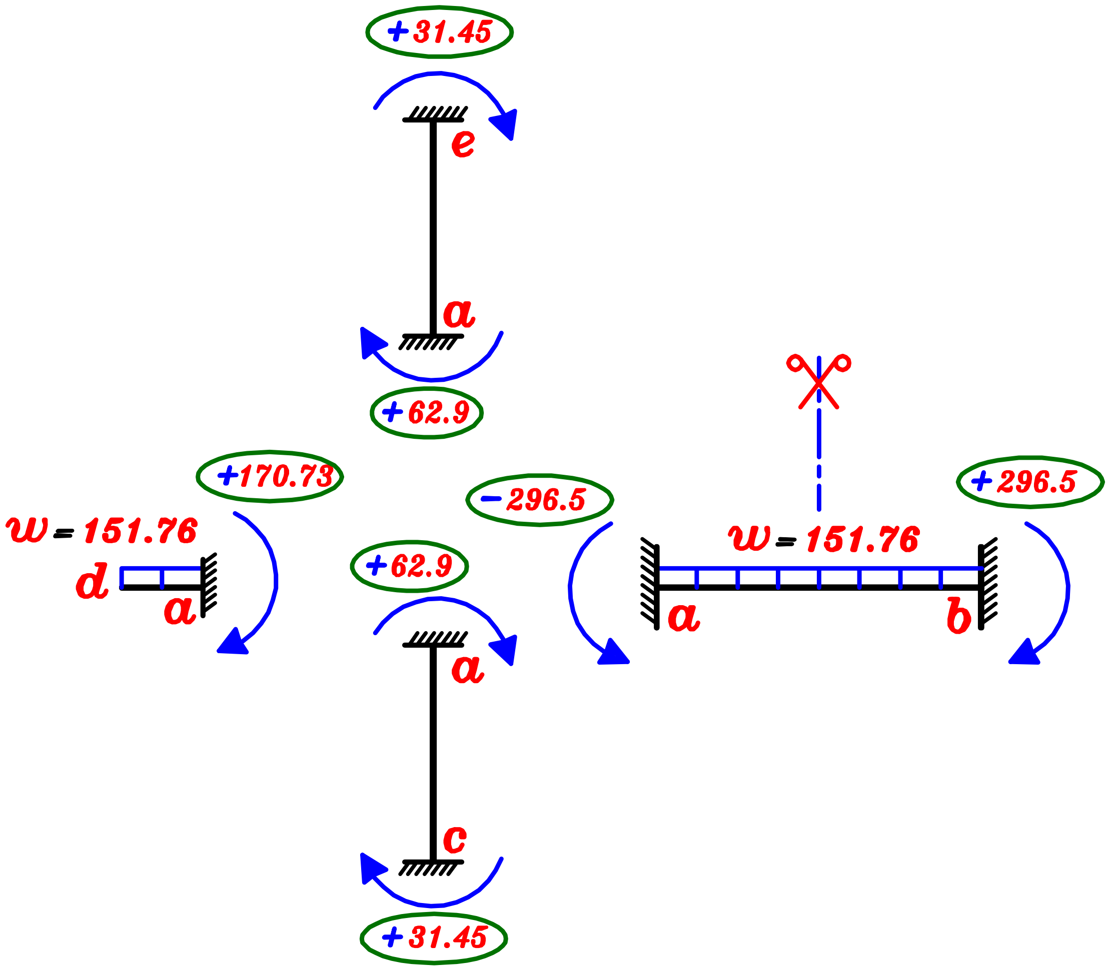
$$F.E.M.(ab) = -\frac{wL^2}{12} = -\frac{151.76 * 6.0^2}{12} = -455.28 \ \text{kN.m.}$$

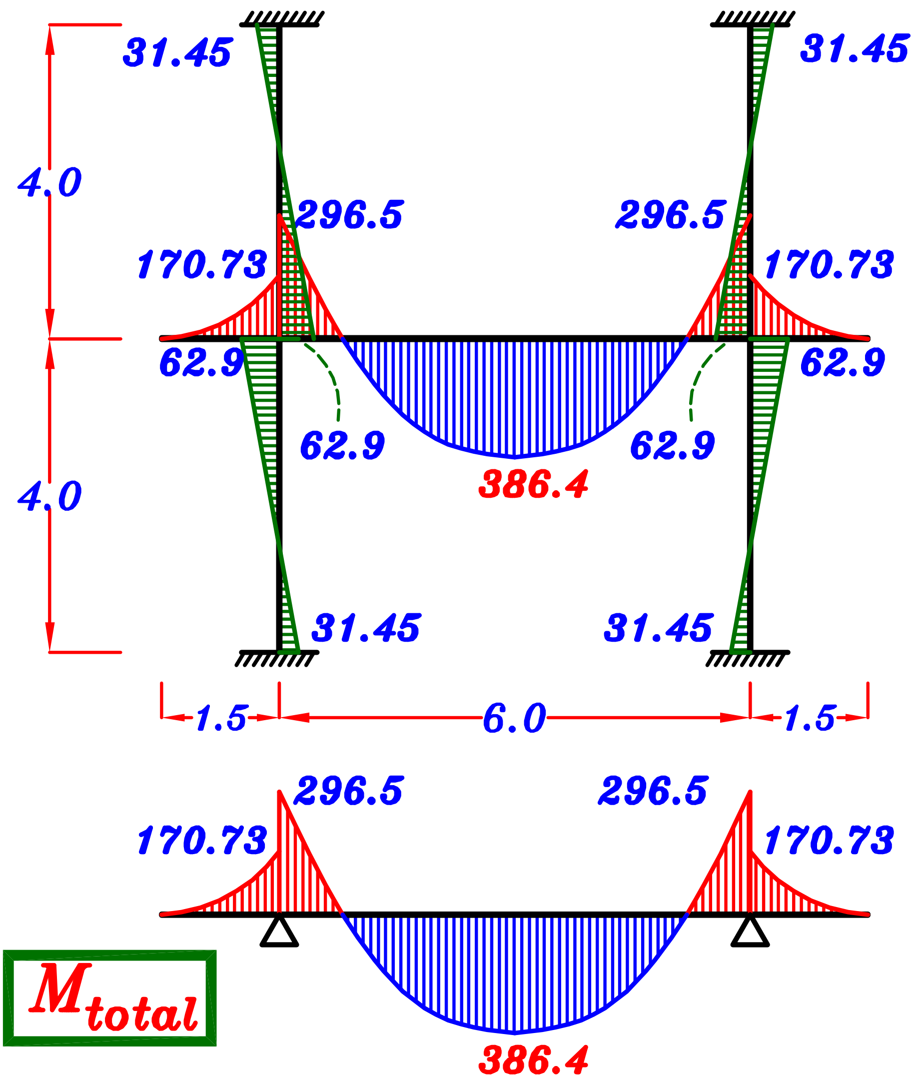
$$F.E.M.(ba) = +\frac{wL^2}{12} = +\frac{151.76 * 6.0^2}{12} = +455.28 \ \text{kN.m.}$$

$$F.E.M.(ad) = +\frac{wL^2}{2} = +\frac{151.76 * 1.5^2}{2} = +170.73 \ \text{kN.m.}$$



Joint	c	a				e
member	c−a	a−c	a−d	a−b	a−e	e−a
D.F.	0	0.221	0	0.558	0.221	0
F.E.M.	0	0	+170.73	−455.28	0	0
B.M.	0	+62.9	0	+158.78	+62.9	0
C.O.M.	+31.45	0	0	0	0	+31.45
B.M.	0	0	0	0	0	0
M_F	+31.45	+62.9	+170.73	−296.5	+62.9	+31.45


+31.45
e
a
+62.9
+170.73
w=151.76
d
a
+62.9
a
c
+31.45
-296.5
w=151.76
a
b
+296.5

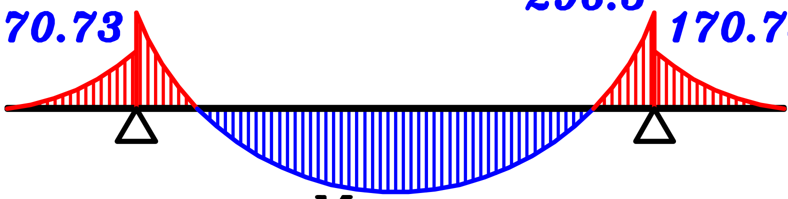

31.45
31.45
4.0
296.5
296.5
170.73
170.73
62.9
62.9
62.9
62.9
386.4
4.0
31.45
31.45
1.5
6.0
1.5
296.5
296.5
170.73
170.73
M total
386.4

Distribute the moment of the Frame on Column Strip and Field Strip.

$\boxed{M_{total}}$

$M_2 = 296.5$

$M_1 = 170.73$ — 296.5 — 170.73



$M_2 = 386.4$

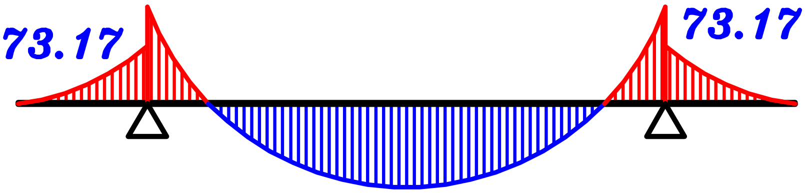
عرض ال C.S.

$M_1 * \frac{3.0}{7.0} = 73.17$

عرض الشريحه كله

moment C.S.

$0.80\ M_2 = 237.2$ — 237.2 — 73.17



$0.55\ M_3 = 212.52$

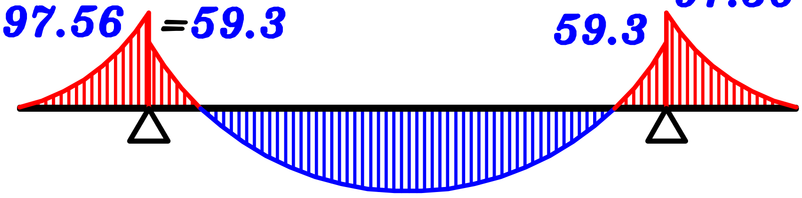
عرض ال F.S.

$M_1 * \frac{4.0}{7.0} = 97.56$

عرض الشريحه كله

moment F.S.

$0.20\ M_2 = 59.3$ — 59.3 — 97.56



$0.45\ M_3 = 173.88$

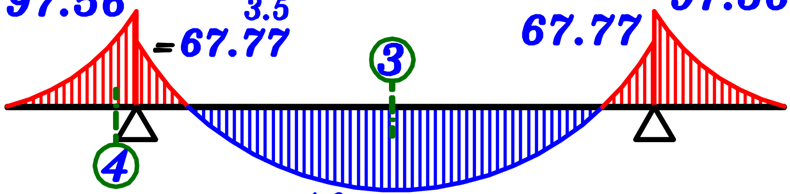
$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{4.0}{3.5}$$

$$\boxed{(M_{F.S})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.0}{3.5}}$$

moment F.S.

$b_{F.S.} = 4.0\ m$

97.56 — $59.3 * \frac{4.0}{3.5} = 67.77$ — 67.77 — 97.56



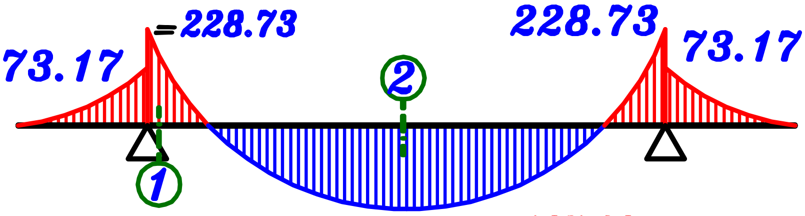
$173.88 * \frac{4.0}{3.5} = 198.72$

$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

moment C.S.

$b_{C.S.} = 3.0\ m$

73.17 — $237.2 + 59.3 - 67.77 = 228.73$ — 228.73 — 73.17



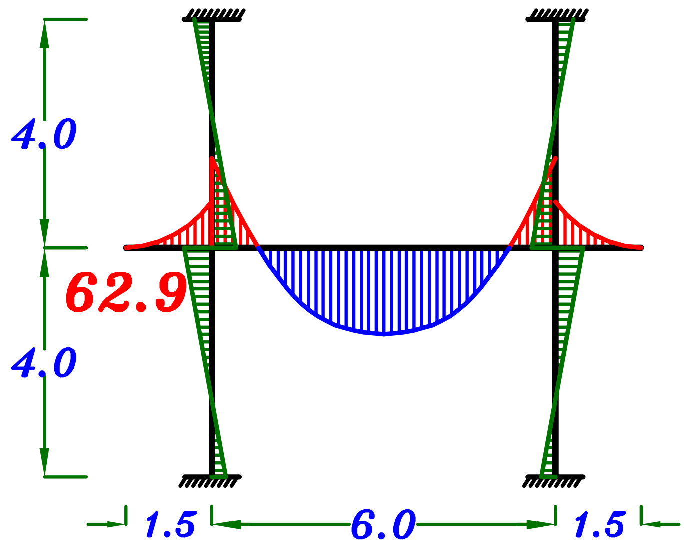
$212.52 + 173.88 - 198.72 = 187.68$

Design of sections. $d = t_s - 40\ mm$

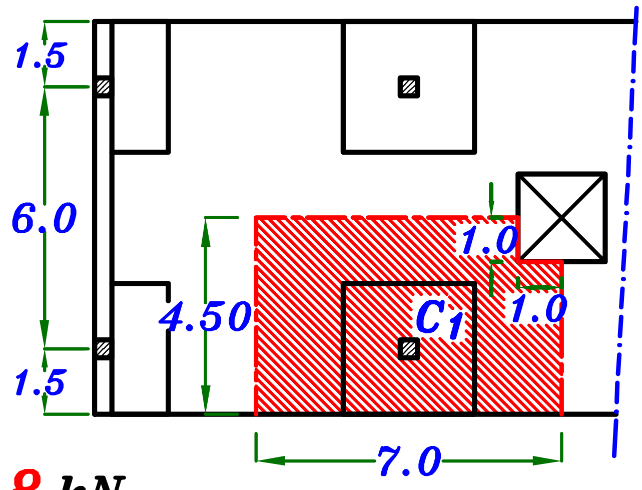
Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm^2/b)	A_s (mm^2/m)	No. of bars/m
Column Strip	1	228.73	3000	260	4.71	0.822	2973	991	5∅16\m
	2	187.68	3000	160	3.19	0.760	4287	1429	8∅16\m
Field Strip	3	198.72	4000	160	3.59	0.786	4389	1097	6∅16\m
	4	97.56	4000	160	5.12	0.826	2050	512.5	5∅12\m

③ Design the column C_1 at ground Floor.

$$\boxed{M_{ext} = 62.9\ kN.m}$$



$$P\backslash Floor = w_s * area * 1.1$$



$$P\backslash Floor =$$

$$= 21.68\ [7.0 * 4.5 - 1.0 * 1.0] * 1.1$$

$$= 727.36\ kN$$

عدد الادوار

$$P_{(total)} = 727.36 * 5.0 = 3636.8\ kN$$

$$\boxed{M_{ext} = 62.9\ kN.m} \qquad \boxed{P_{total} = 3636.8\ kN}$$

لتحديد أبعاد العمود المبدئيه نفرض أن العمود عليه *axial load* فقط

$$P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y \xrightarrow{Take} \mu = \frac{A_s}{cA} = 1.0\ \%$$

$$\therefore 3636.8 * 10^3 = 0.35\,(A_c)(25) + 0.67\left(\frac{A_c}{100}\right)(360)$$

$$\longrightarrow A_c = 325819\ mm^2 \longrightarrow b = \sqrt{A_c} = \sqrt{325819} = 570.8\ mm$$

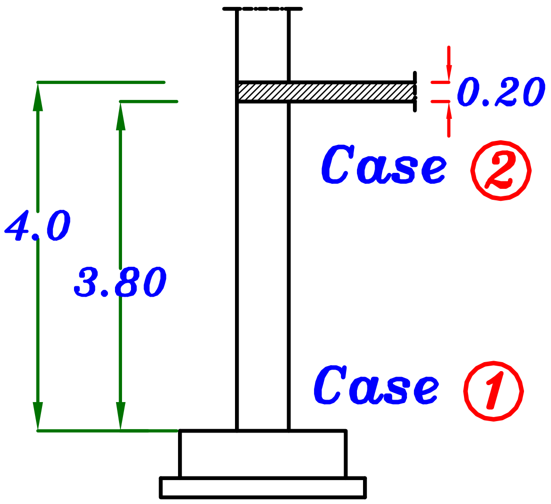
Take $\boxed{b = 600\ mm}$

يجب أن لا تقل b عن $350\ mm$
حتى تكون البلاطه *Safe Punching* .

Check Buckling.

In plane & out of plane.

العمود متماثل فى الاتجاهين



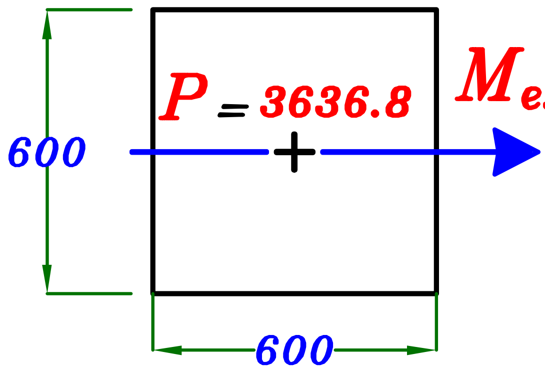
Upper Case ②
Lower Case ①

$$\left.\begin{matrix}\text{Upper Case } 2\\ \text{Lower Case } 1\end{matrix}\right\} k = 1.3$$

$$H_o = 3.80\ m$$

$$\lambda_b = \frac{1.3 * 3.80}{0.60} = 8.23 < 10$$

$$\lambda_b < 10 \longrightarrow \text{Short Column.} \longrightarrow \text{NO } M_{add}$$



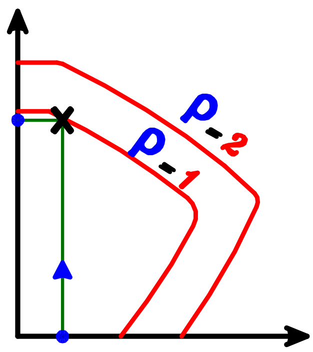
$$M_{ext} = 62.9\ kN.m$$

$$e = \frac{M}{P} = \frac{62.9}{3636.8} = 0.017\ m$$

$$\frac{e}{t} = \frac{0.017}{0.60} \simeq 0.028 \xrightarrow{use} I.D.$$

$$\zeta = \frac{600\ -100}{600} = 0.83 \xrightarrow{use} \text{ECCS Design Aids Page } 4\text{–}24$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{3636.8 * 10^3}{25 * 600 * 600} = 0.404 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{62.9 \quad * 10^6}{25 * 600 * 600^2} = 0.011 \end{aligned}\right\} \rho = 1.0$$



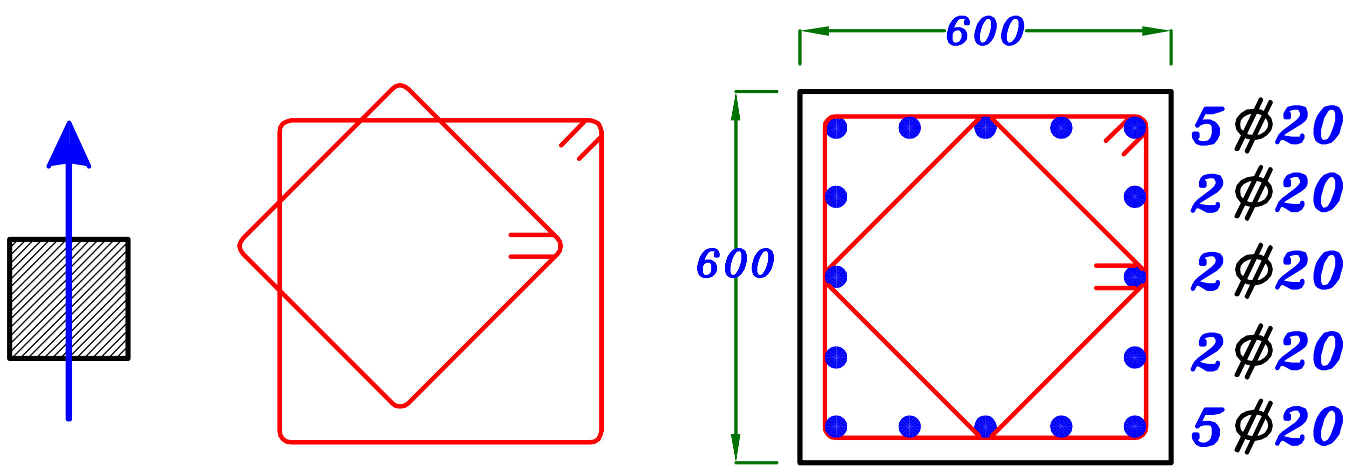
$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 25 * 10^{-4} = 2.5 * 10^{-3}$$

$$A_s = A_s` = \mu * b * t = 2.5 * 10^{-3} * 600 * 600 = 900\ mm^2$$

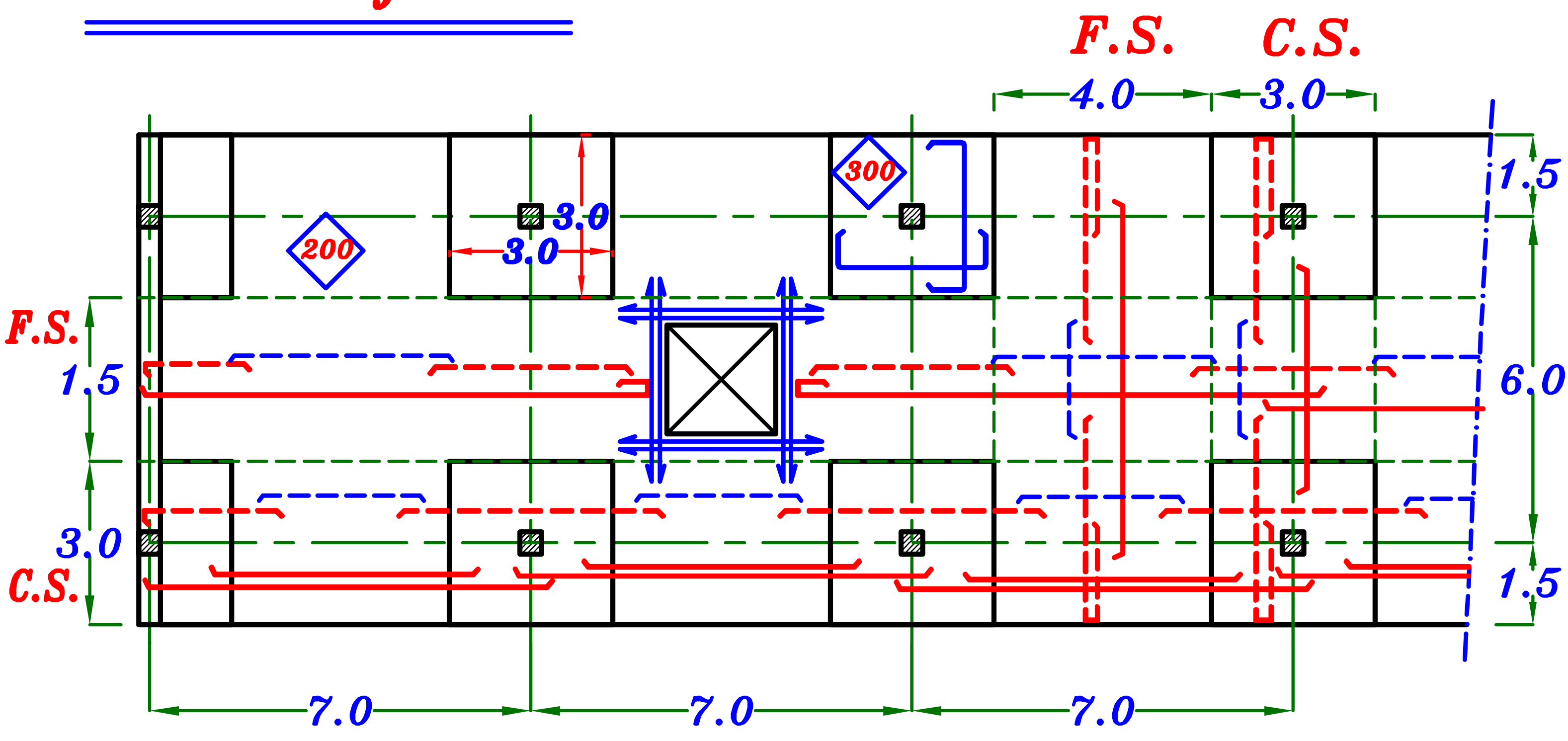
$$A_{s\,Total} = A_s + A_s` = 2 * 900 = 1800\ mm^2$$

$$A_{s\,min} = \frac{0.8}{100} * b * t = \frac{0.8}{100} * 600 * 600 = 2160\ mm^2 > A_{s\,total}$$

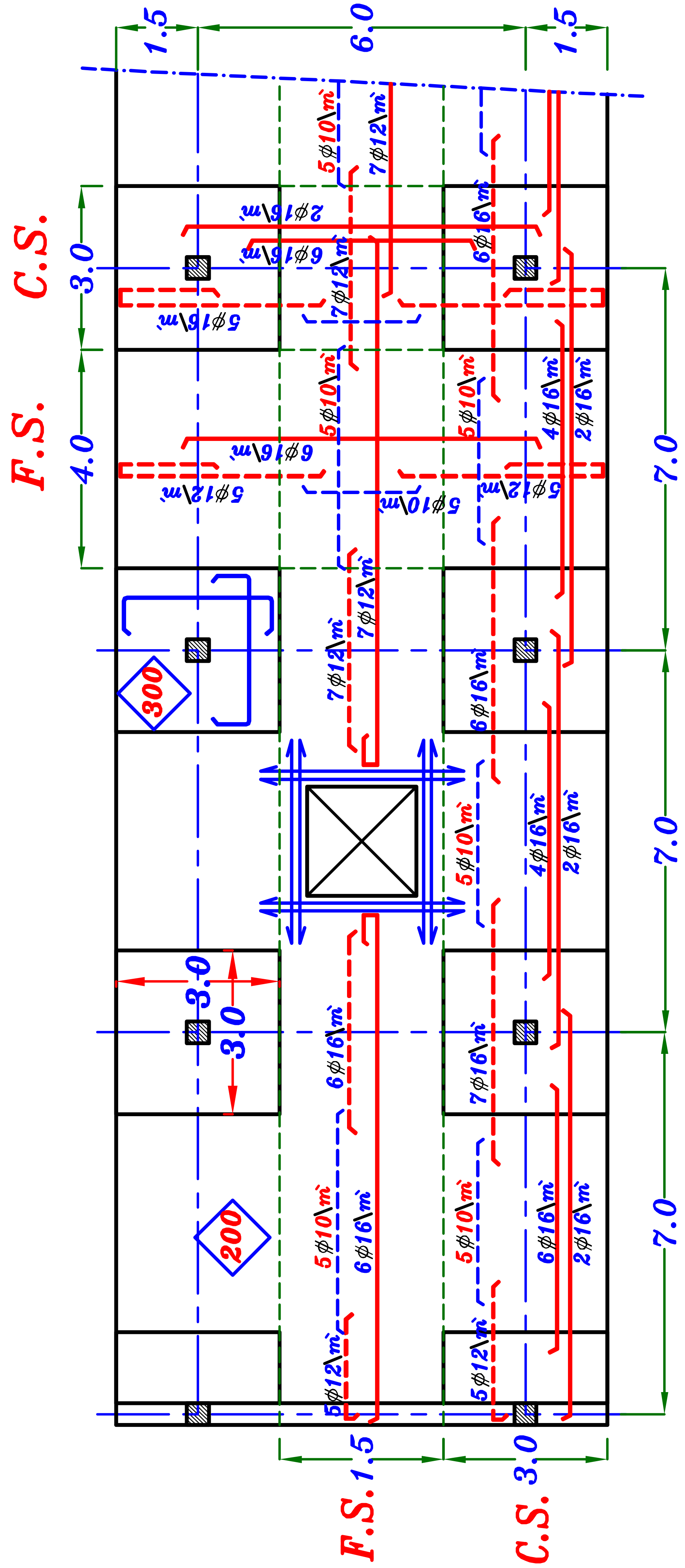
$$\text{Take } A_s = A_s` = \frac{A_{s\,min}}{2} = 1080\ mm^2 \quad \boxed{5 \phi 20}$$



Details of RFT.



Details of RFT.



Example.

The given plan shows general layout of one storey Flat slab Roof.

Data. $F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

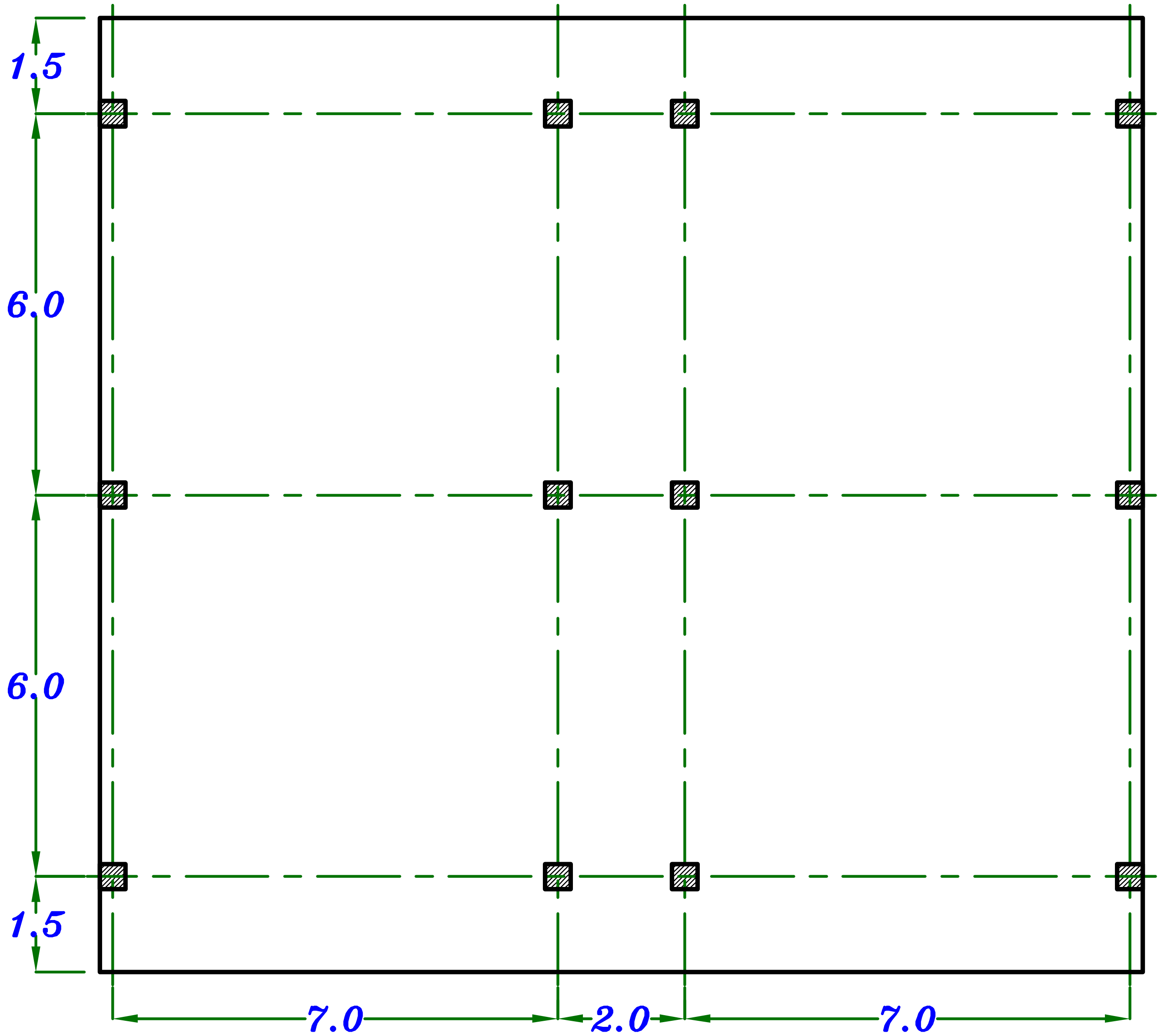
$F.C. = 2.5\ kN\backslash m^2$, $L.L. = 3.0\ kN\backslash m^2$

The Floor height = 4.0 m

Req.

② Complete design of the slab in both directions.

③ Draw details of reinforcement in plan.



Solution.

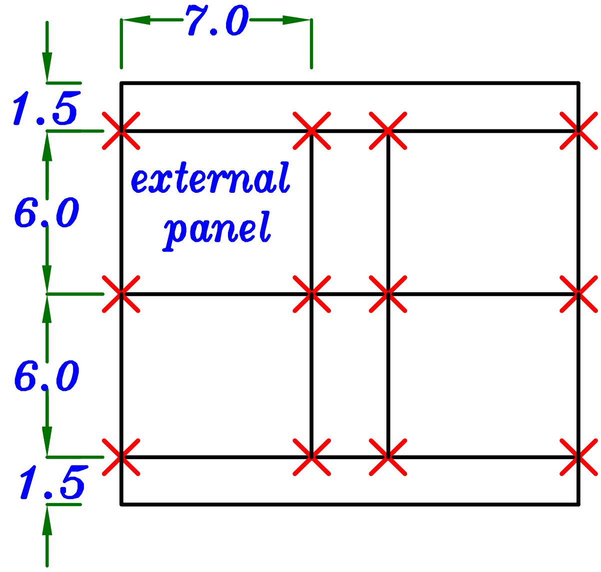
1-Concrete Dimensions.

Column dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \frac{H}{15} = \frac{4000}{15} = 266.6\ mm \\ \frac{L_1}{20} = \frac{7000}{20} = 350\ mm \end{cases}$$

$$\boxed{b_{Col.} = 350\ mm}$$

$(350 * 350)$



Slab Thikness.

$$L_1 = 7.0\ m$$

External panel $t_s = \frac{L_1}{32} = \frac{7000}{32} = 218.7\ mm$

Cantilever $t_s = \frac{L_c}{10} = \frac{1500}{10} = 150\ mm$

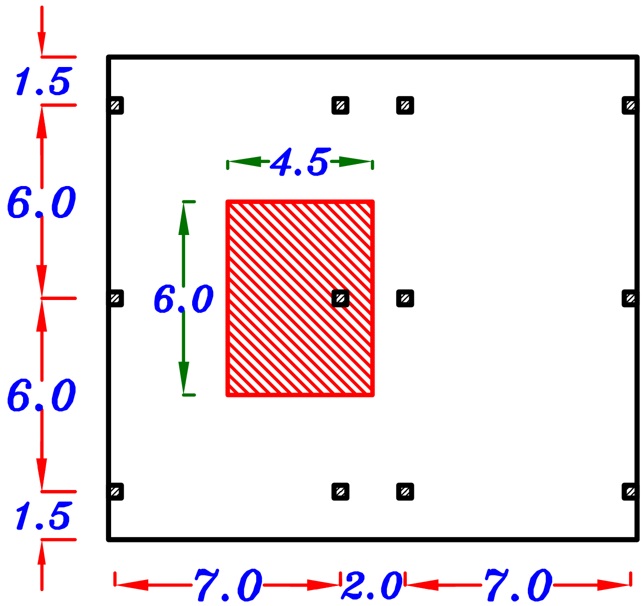
$$\boxed{t_s = 220\ mm}$$

2-Loads on the Slab.

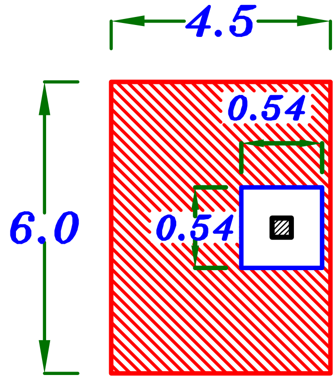
$$w_s = 1.4\ (t_s\ \gamma_c + F.C.) + 1.6\ (L.L.)$$

$$w_s = 1.4\ (0.22 * 25 + 2.50) + 1.6\ (3.0) = 16.0\ kN\backslash m^2$$

Check Punching on interior column C_1



$\underline{\underline{C_1}}$ Interior Column.



$$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$$

$$C + d = 0.35 + 0.19 = 0.54\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 16.0\ \left[6.0 * 4.5 - 0.54 * 0.54\right] = 427.33\ kN$$

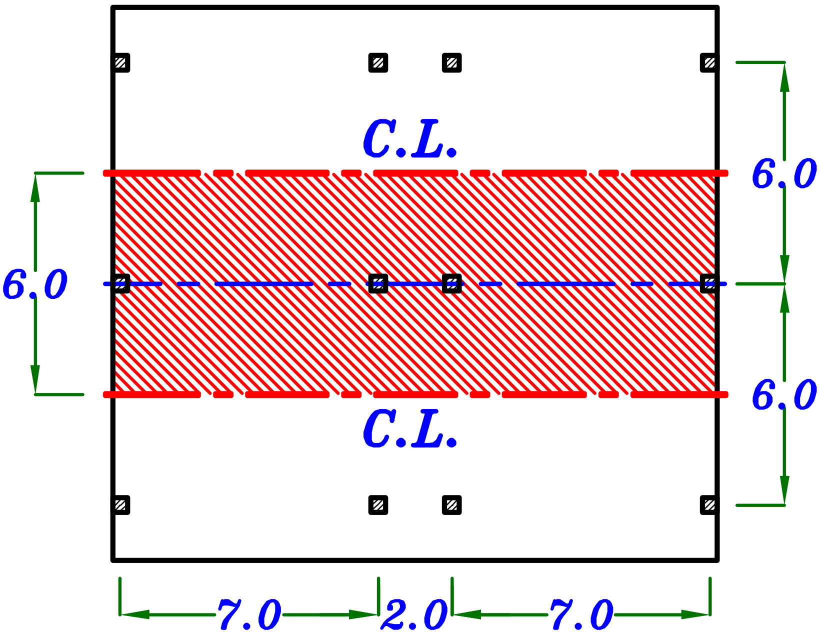
$$A_p = (b_o * d) = (4 * 540) * 190 = 410400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{427.33 * 10^3}{410400} * 1.15 = 1.19\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\delta_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.

Long Direction.



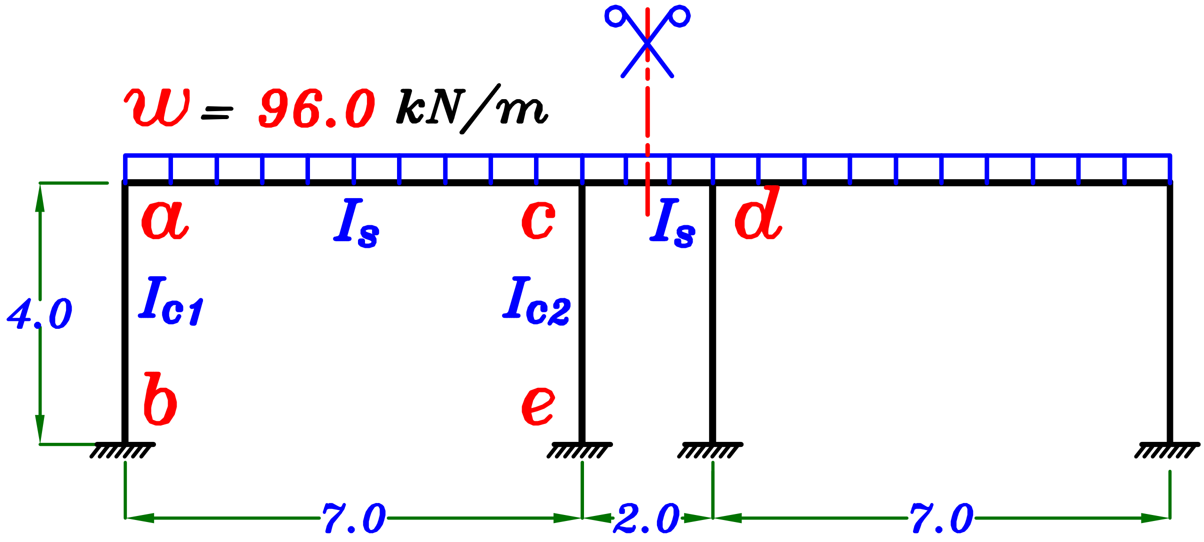
$$Span = L_1 = 7.0\ m$$

$$Width = L_2 = 6.0\ m$$

$$b_{C.S.} = \frac{L_2}{2} = 3.0\ m$$

$$b_{F.S.} = L_2 - \frac{L_2}{2} = 3.0\ m$$

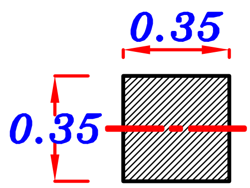
$$w = w_s * L_2 = 16.0 * 6.0 = 96.0\ kN/m$$



@ **Calculate Moment of Inertia For Slabs & Columns.**

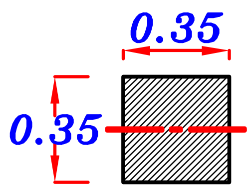
عمود خارجى

$$I_{c1} = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.35 * 0.35^3}{12} = 7.50 * 10^{-4}\ m^4$$

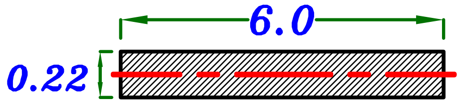


عمود داخلى

$$I_{c2} = 0.3 * \frac{b(t)^3}{12} = 0.3 * \frac{0.35 * 0.35^3}{12} = 3.75 * 10^{-4}\ m^4$$



$$I_s = \frac{L_2 * t_s^3}{12} = \frac{6.0 * 0.22^3}{12} = 5.32 * 10^{-3}\ m^4$$



ⓑ **Calculate the stiffness For each member.**

$$K_{ac} = \frac{I_s}{L} = \frac{5.32 * 10^{-3}}{7.0} = 7.60 * 10^{-4} = K_{ca}$$

$$K_{cd} = \frac{1}{2} * \frac{I_s}{L} = \frac{1}{2} * \frac{5.32 * 10^{-3}}{2.0} = 1.33 * 10^{-3}$$

$$K_{ab} = \frac{I_c}{h} = \frac{7.50 * 10^{-4}}{4.0} = 1.875 * 10^{-4}$$

$$K_{ce} = \frac{I_c}{h} = \frac{3.75 * 10^{-4}}{4.0} = 9.375 * 10^{-5}$$

ⓒ **Calculate the Distribution Factors. (D.F.)**

For Joint a

$$\Sigma K = K_{ab} + K_{ac} = 1.875 * 10^{-4} + 7.60 * 10^{-4} = 9.475 * 10^{-4}$$

$$D.F._{ab} = \frac{1.875 * 10^{-4}}{9.475 * 10^{-4}} = 0.20$$

$$D.F._{ac} = \frac{7.60 * 10^{-4}}{9.475 * 10^{-4}} = 0.80$$

For Joint C

$$\Sigma K = K_{ca} + K_{ce} + K_{cd} = 7.60 * 10^{-4} + 9.375 * 10^{-5} + 1.33 * 10^{-3} = 2.18 * 10^{-3}$$

$$D.F._{ca} = \frac{7.60 * 10^{-4}}{2.18 * 10^{-3}} = 0.348$$

$$D.F._{ce} = \frac{9.375 * 10^{-5}}{2.18 * 10^{-3}} = 0.043$$

$$D.F._{cd} = \frac{1.33 * 10^{-3}}{2.18 * 10^{-3}} = 0.609$$

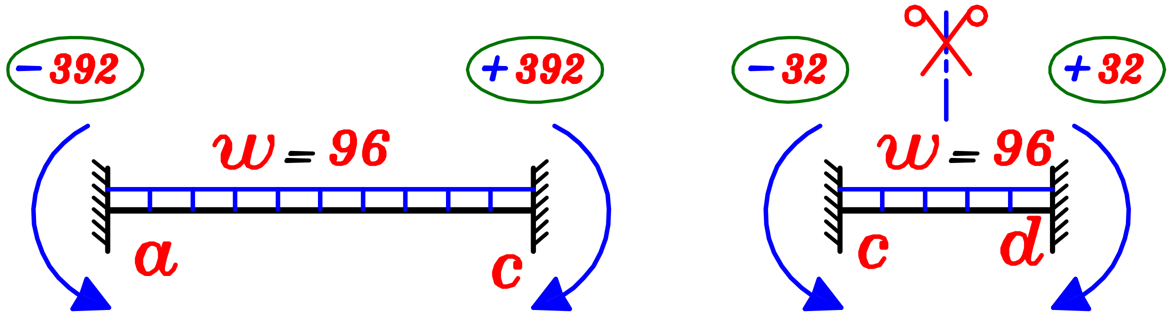
ⓓ Calculate Fixed End Moment For the Slab.

$$F.E.M.(ac) = -\frac{wL^2}{12} = -\frac{96.0 * 7.0^2}{12} = -392 \text{ kN.m.}$$

$$F.E.M.(ca) = +\frac{wL^2}{12} = +\frac{96.0 * 7.0^2}{12} = +392 \text{ kN.m.}$$

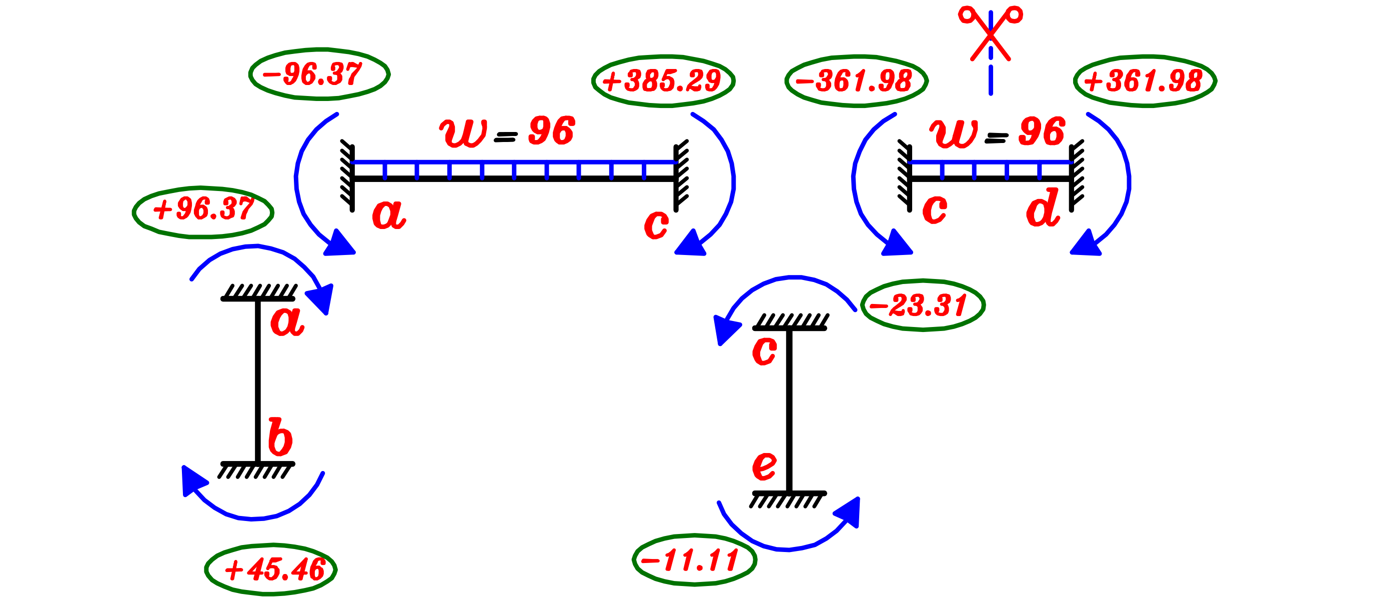
$$F.E.M.(cd) = -\frac{wL^2}{12} = -\frac{96.0 * 2.0^2}{12} = -32 \text{ kN.m.}$$

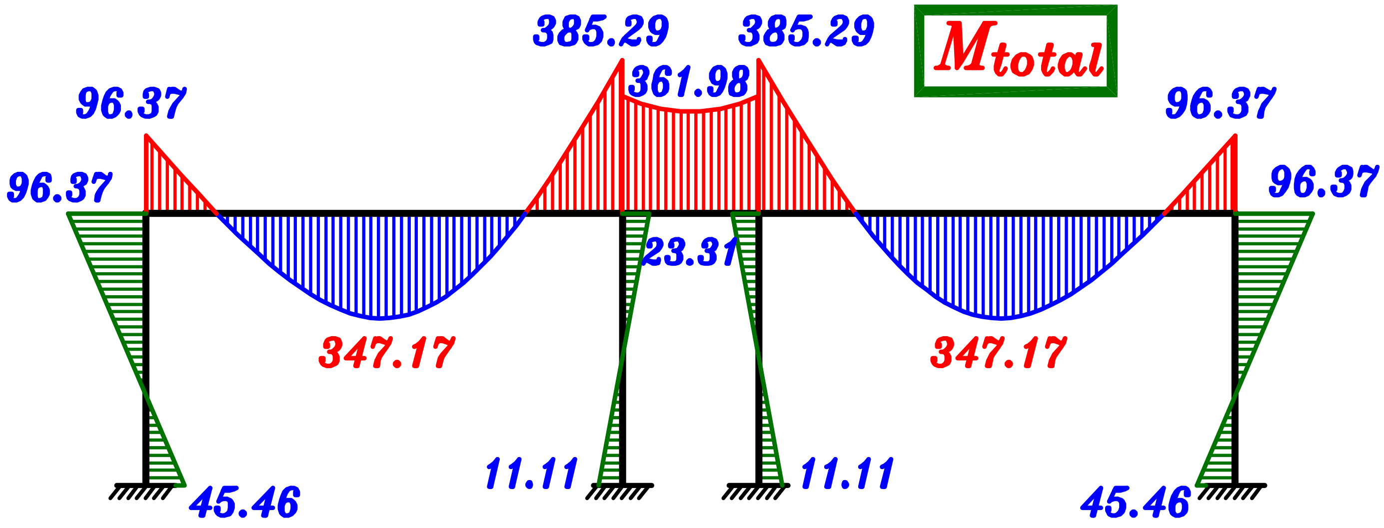
$$F.E.M.(dc) = +\frac{wL^2}{12} = +\frac{96.0 * 2.0^2}{12} = +32 \text{ kN.m.}$$



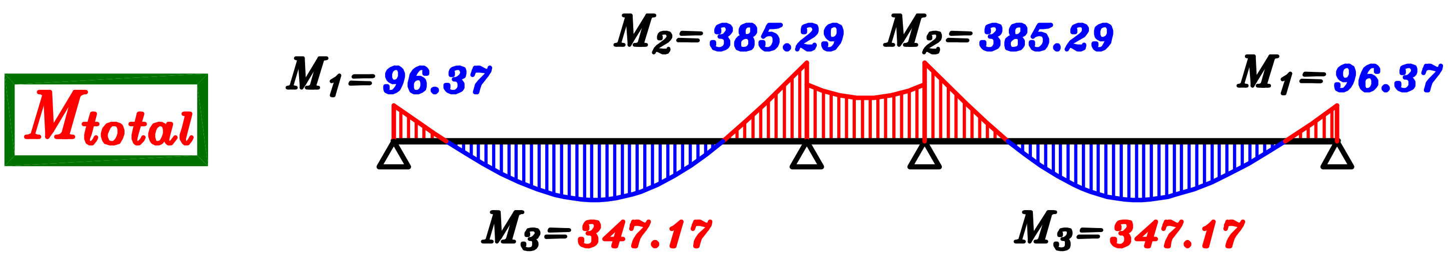
Joint	b	a		c			e
member	b-a	a-b	a-c	c-a	c-d	c-e	e-c
D.F.	0	0.20	0.80	0.348	0.609	0.043	0
F.E.M.	0	0	-392	+392	-32	0	0
B.M.	0	+78.4	+313.6	-125.28	-219.24	-15.48	0
C.O.M.	+39.2	0	-62.64	+156.8	0	0	-7.74
B.M.	0	+12.52	+50.11	-54.56	-95.49	-6.742	0
C.O.M.	+6.26	0	-27.28	+25.05	0	0	-3.371
B.M.	0	+5.45	+21.83	-8.717	-15.25	-1.077	0
M_F	+45.46	+96.37	-96.37	+385.29	-361.98	-23.30	-11.11

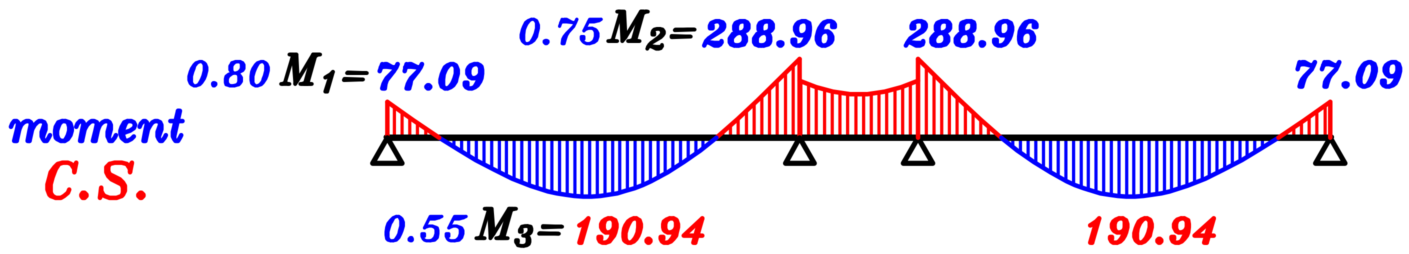
كفايه عمل دورتين فى ال moment distribution

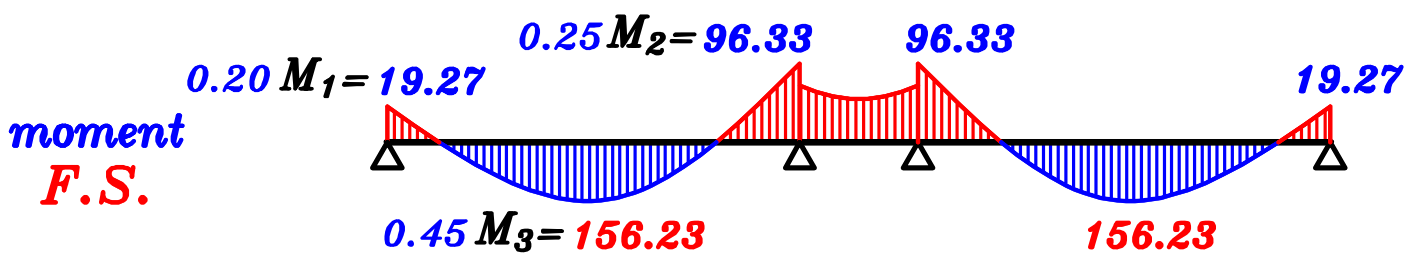




5−Distribute the moment of the Frame on Column Strip and Field Strip.

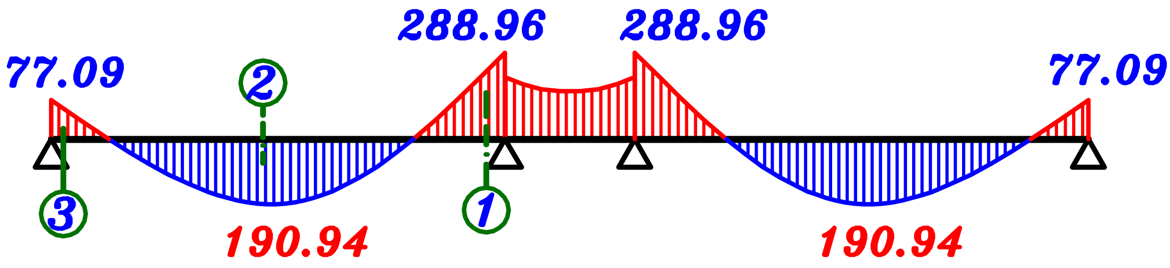




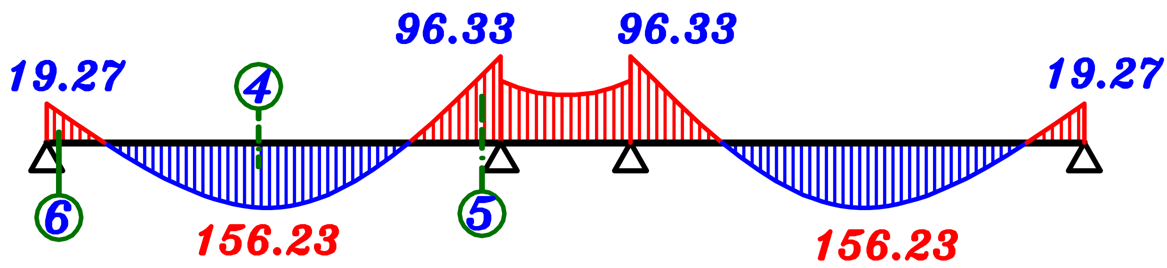


6- Design the sections of the slab.

moment
C.S.
$b_{F.S.} = 3.0\ m$



moment
F.S.
$b_{F.S.} = 3.0\ m$



Design of sections.

$d = t_s - 30\ mm = 220 - 30 = 190\ mm$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm^2/b)	As (mm^2/m)	No. of bars/m
Column Strip	1	288.96	3000	190	3.06	0.747	4941	1647	7Ø18\m
	2	190.94	3000	190	3.76	0.794	2872	957	5Ø16\m
	3	77.09	3000	190	5.92	0.826	1654	551	5Ø12\m
Field Strip	4	156.23	3000	190	4.16	0.808	1503	501	5Ø12\m
	5	96.33	3000	190	5.30	0.826	1353	451	5Ø12\m
	6	19.27	3000	190	11.85	0.826	341	113	5Ø12\m

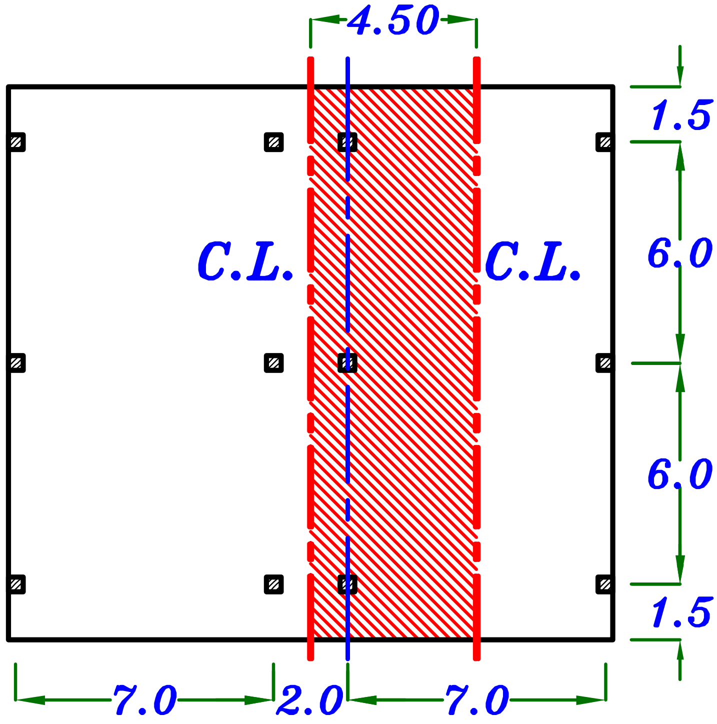
Short Direction.

$Span = 6.0\ m$

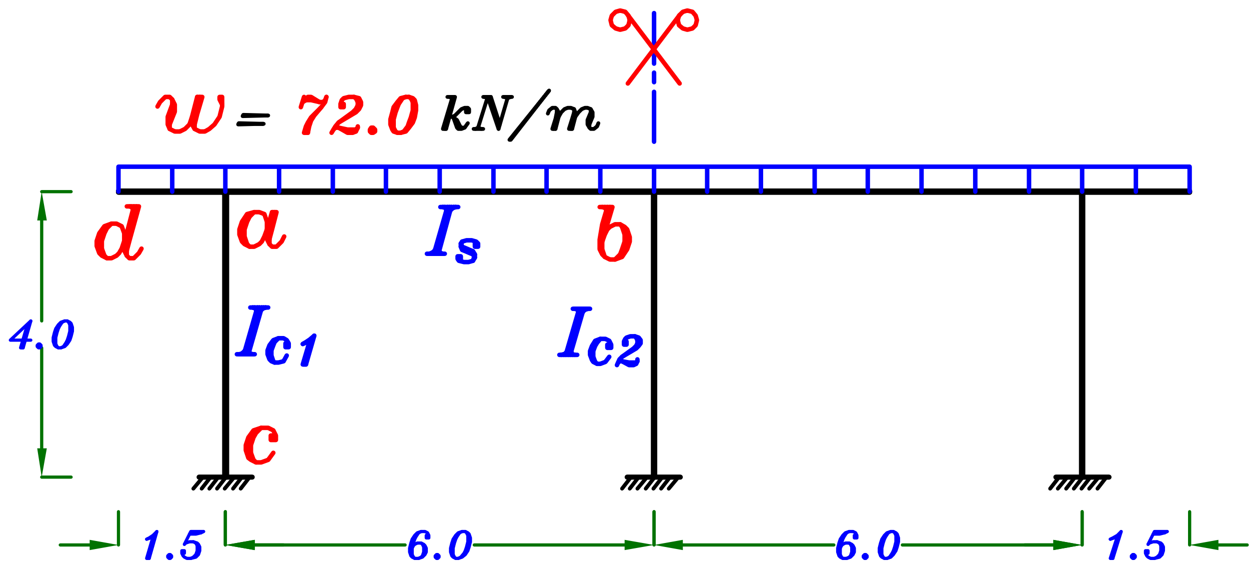
$Width = 4.5\ m$

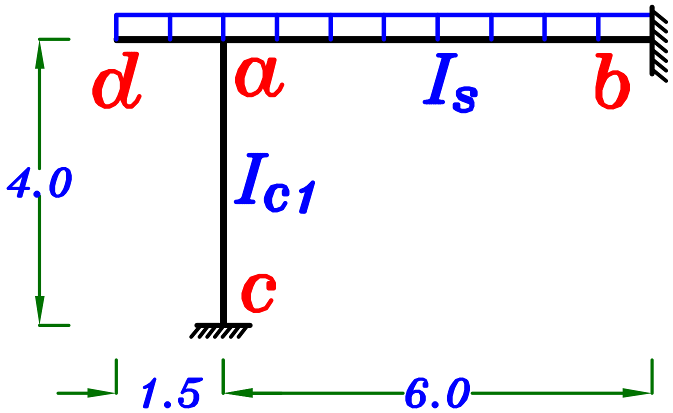
$b_{C.S.} = 1.0 + \frac{6.0}{4} = 2.50\ m$

$b_{F.S.} = 4.5 - 2.0 = 2.0\ m$



$w = w_S * L_2 = 16.0 * 4.5 = 72.0\ kN/m$

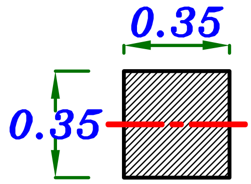




ⓐ Calculate Moment of Inertia For Slabs & Columns.

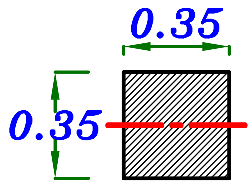
عمود خارجي

$$I_{c1} = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.35 * 0.35^3}{12} = 7.50 * 10^{-4}\ m^4$$

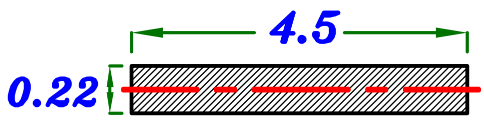


عمود داخلي

$$I_{c2} = 0.3 * \frac{b(t)^3}{12} = 0.3 * \frac{0.35 * 0.35^3}{12} = 3.75 * 10^{-4}\ m^4$$



$$I_S = \frac{L_2 * t_s^3}{12} = \frac{4.5 * 0.22^3}{12} = 3.99 * 10^{-3}\ m^4$$



ⓑ Calculate the stiffness For each member.

$$K_{ab} = \frac{I_s}{L} = \frac{3.99 * 10^{-3}}{6.0} = 6.65 * 10^{-4}$$

$$K_{ac} = \frac{I_{c1}}{h} = \frac{7.50 * 10^{-4}}{4.0} = 1.875 * 10^{-4}$$

$$K_{ad} = Zero$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

$$\Sigma K = K_{ab} + K_{ac} = 6.65 * 10^{-4} + 1.875 * 10^{-4} = 8.525 * 10^{-4}$$

$$D.F._{ab} = \frac{6.65 * 10^{-4}}{8.525 * 10^{-4}} = 0.78$$

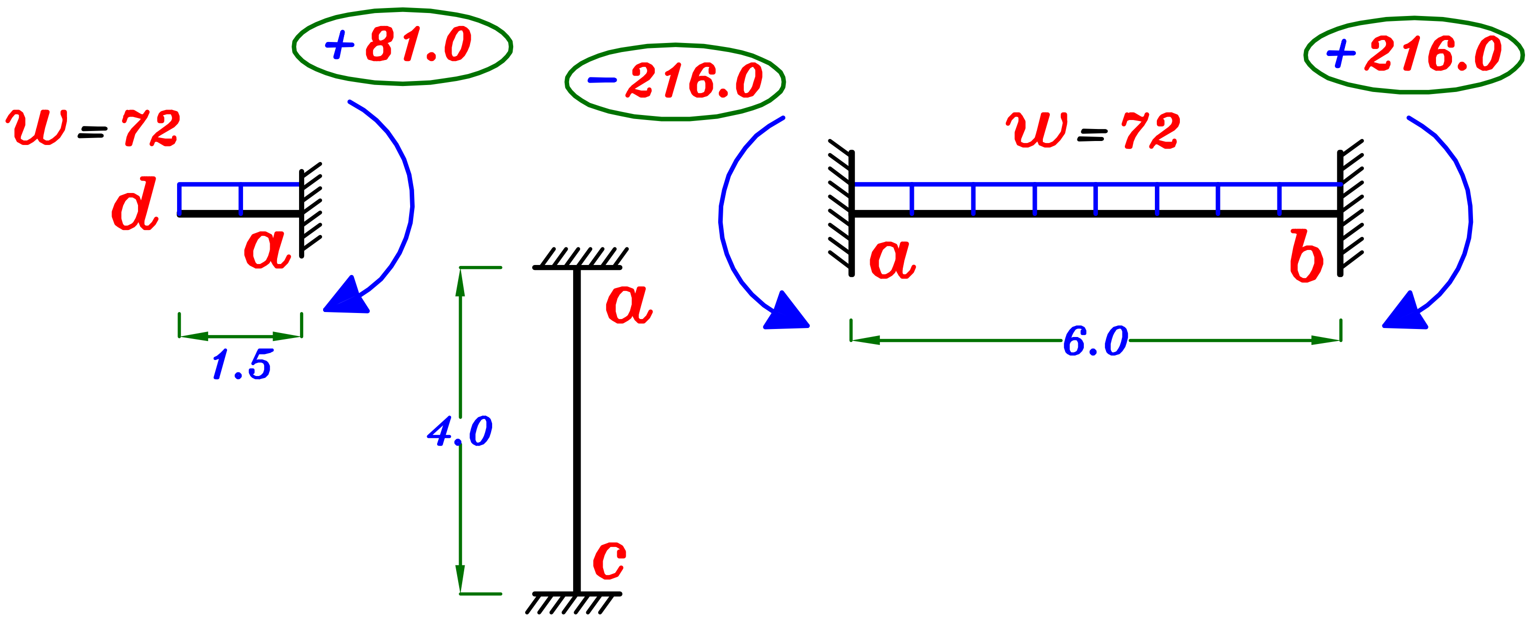
$$D.F._{ac} = \frac{1.875 * 10^{-4}}{8.525 * 10^{-4}} = 0.22$$

ⓓ Calculate Fixed End Moment For the Slab.

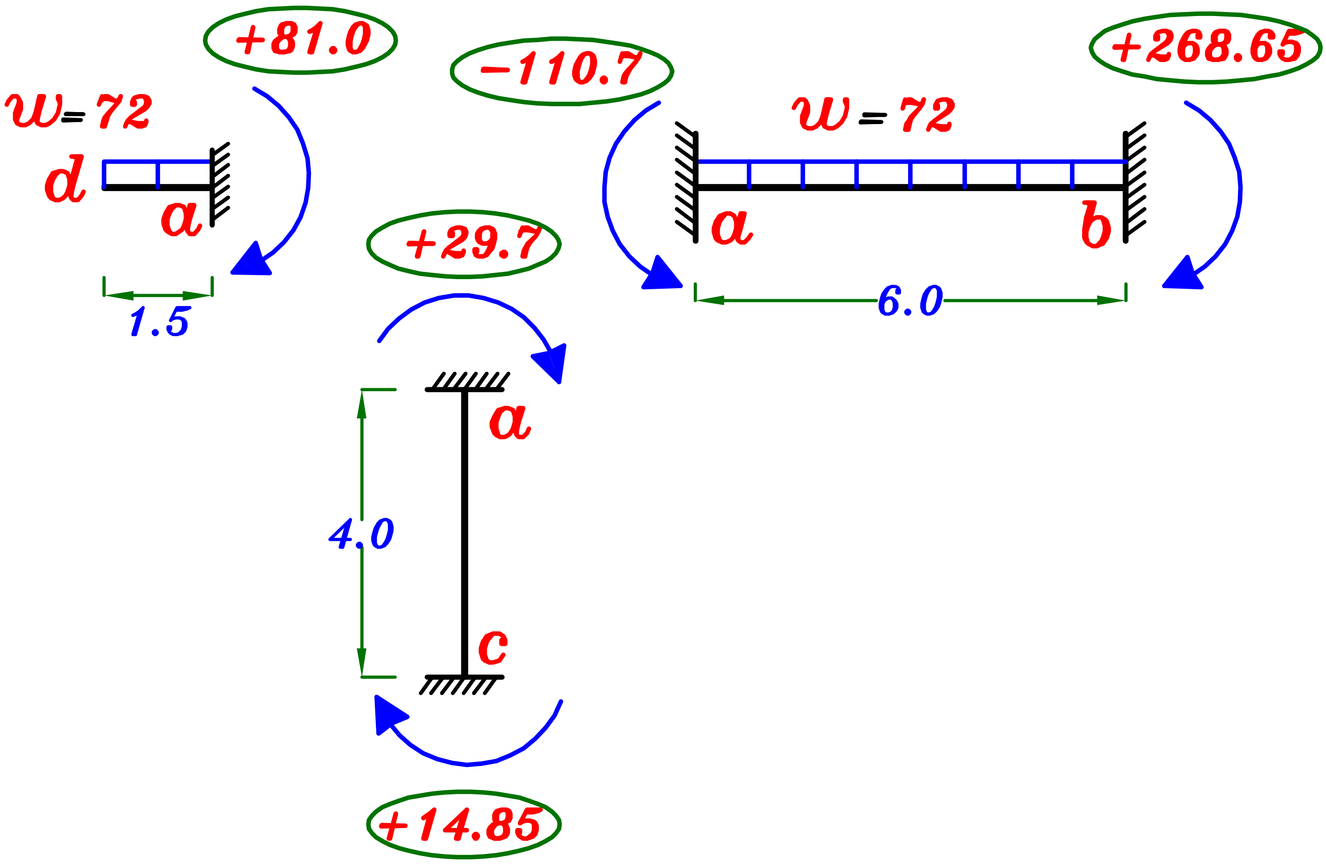
$$F.E.M.(ab) = -\frac{wL^2}{12} = -\frac{72.0 * 6.0^2}{12} = -216.0 \text{ kN.m.}$$

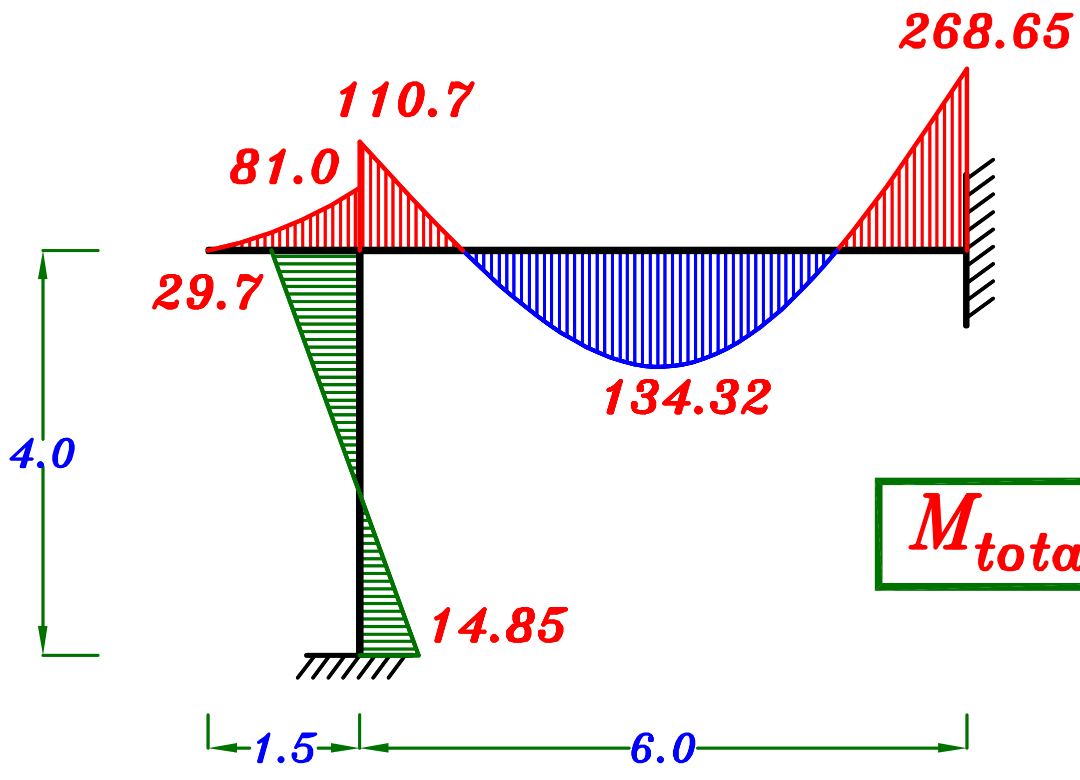
$$F.E.M.(ba) = +\frac{wL^2}{12} = +\frac{72.0 * 6.0^2}{12} = +216.0 \text{ kN.m.}$$

$$F.E.M.(ad) = +\frac{wL^2}{2} = +\frac{72.0 * 1.5^2}{2} = +81.0 \text{ kN.m.}$$

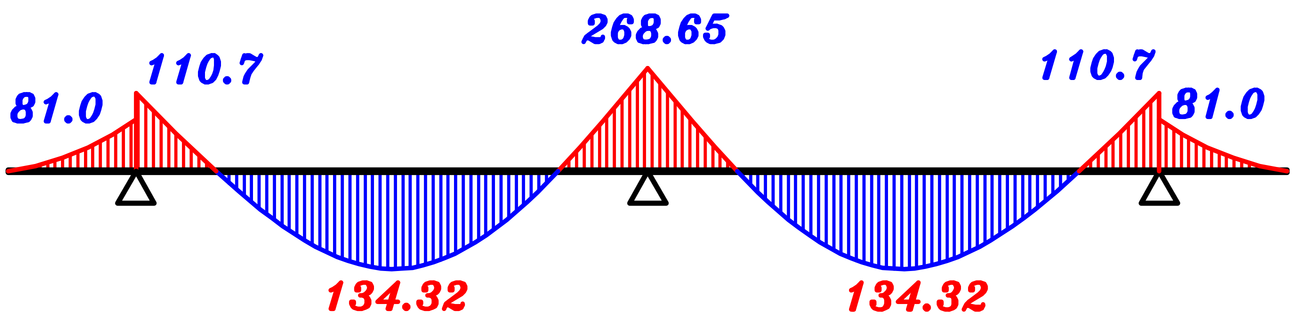


Joint	c	a			b
member	c − a	a − c	a − d	a − b	b − a
D.F.	0	0.22	0	0.78	0
F.E.M.	0	0	+81	−216	+216
B.M.	0	+29.7	0	+105.3	0
C.O.M.	+14.85	0	0	0	+52.65
B.M.	0	0	0	0	0
M_F	+14.85	+29.7	+81.0	−110.7	+268.65


+81.0
w=72
d
a
1.5
-110.7
+268.65
w=72
a
b
6.0
+29.7
a
4.0
c
+14.85


268.65
110.7
81.0
29.7
134.32
4.0
14.85
1.5
6.0

M_{total}


268.65
81.0
110.7
110.7
81.0
134.32
134.32

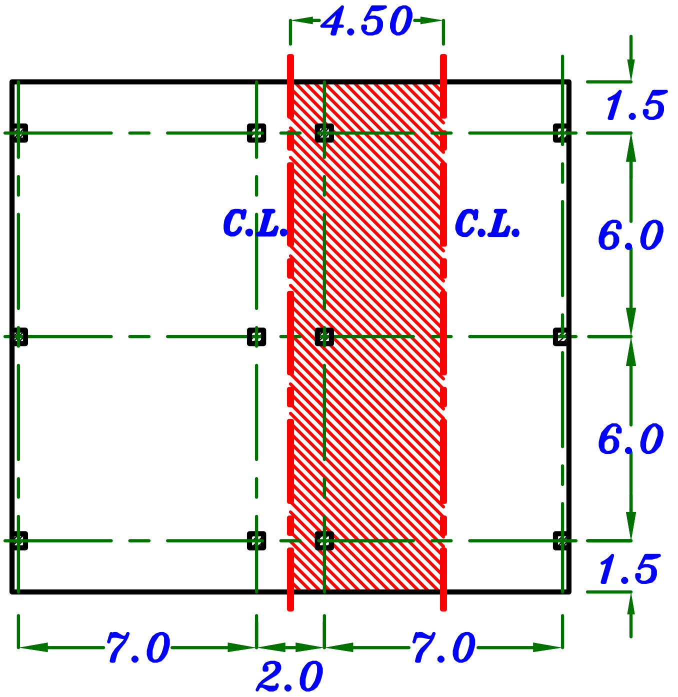
Modification Factor.

يتم أخذ شريحه التصميم الكليه

من C.L. البلاطه الى C.L. البلاطه

فيكون عرض شريحه التصميم الكليه

$$\text{Total Strip width} = \frac{7.0}{2.0} + \frac{2.0}{2.0} = 4.50\ m$$



$$\boxed{b_{C.S.} = \frac{L_2}{4} + \frac{1}{2}\ \text{width of the small span.}}$$

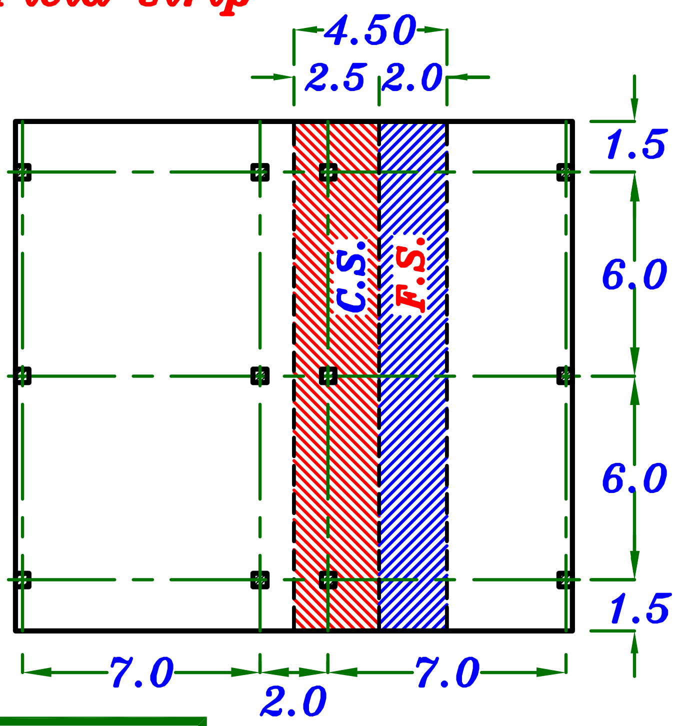
يؤخذ عرض ال **Column strip**

$$b_{C.S.} = \frac{6.0}{4} + \frac{2.0}{2} = 2.50\ m$$

و يؤخذ عرض ال **Field strip**

$$\boxed{b_{F.S.} = \text{Total Strip width} - b_{C.S.}}$$

$$b_{F.S.} = 4.50 - 2.50 = 2.0\ m$$



Modification Factor For Field Strip

$$\boxed{M.F. = \frac{\text{Field Strip العرض الحقيقى لل}}{\text{C.L. to C.L. نصف عرض الشريحه من}}}$$

$$M.F. = \frac{2.0}{\frac{1}{2} * 4.50} = 0.89$$

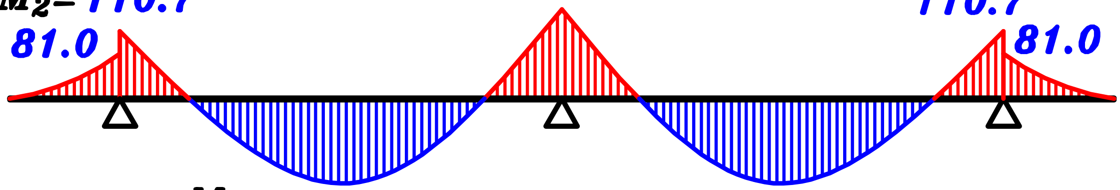
$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor}}$$

ثم يتم اعاده حساب عزم ال F.S. بحيث يظل العزم الكلى ثابت .

$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

5-Distribute the moment of the Frame on Column Strip and Field Strip.

$\boxed{M_{total}}$

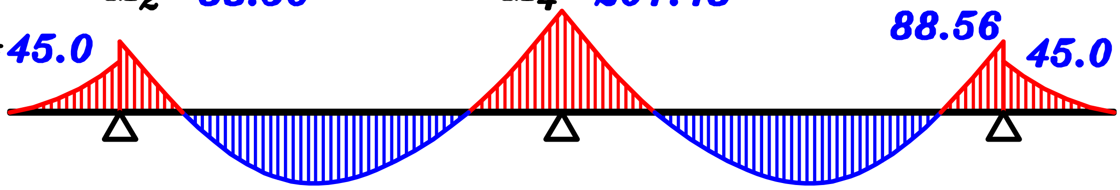


عرض ال C.S.

$M_1 * \frac{2.5}{4.5} = 45.0$

عرض الشريحه كله

moment C.S.

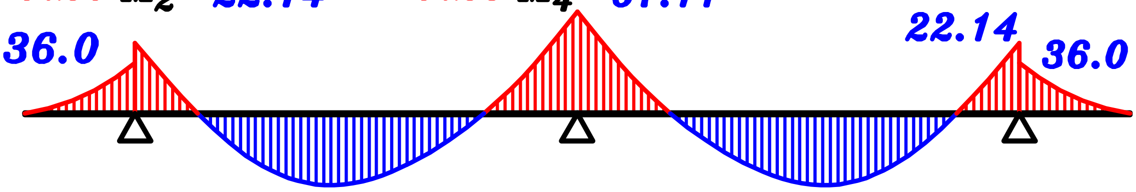


عرض ال F.S.

$M_1 * \frac{2.0}{4.5} = 36.0$

عرض الشريحه كله

moment F.S.

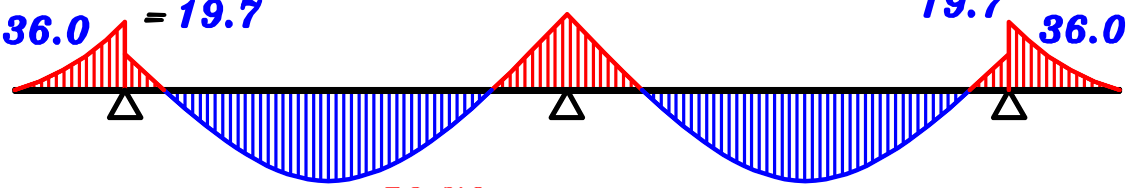


Modification Factor = 0.89

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * 0.89}$$

moment F.S.

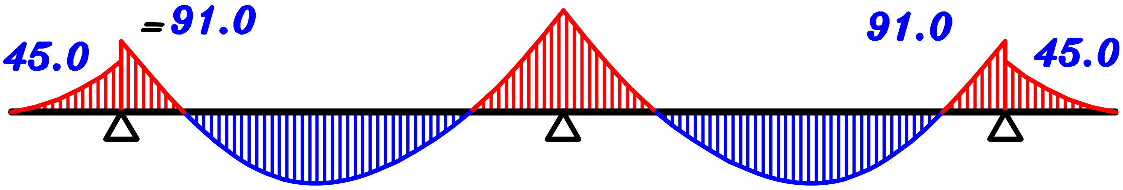
$b_{F.S.} = 2.0\ m$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

moment C.S.

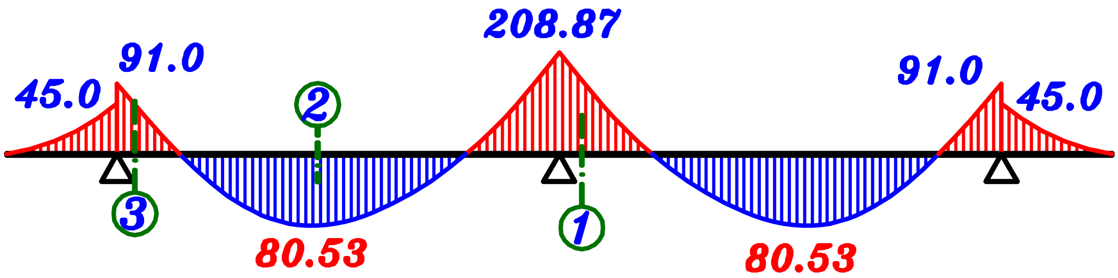
$b_{C.S.} = 2.5\ m$



6– Design the sections of the slab.

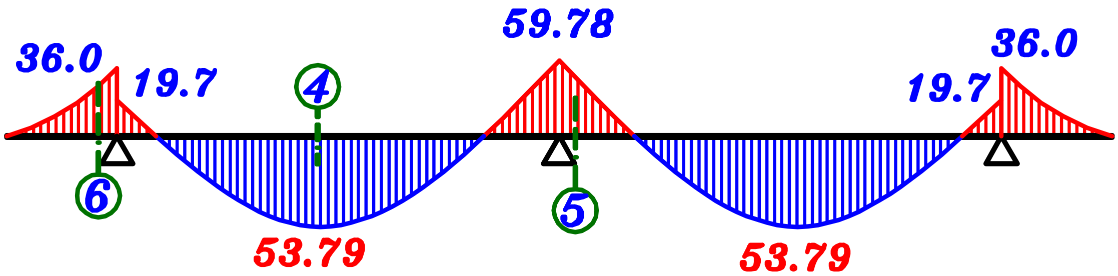
moment C.S.

$b_{C.S.} = 2.5\ m$



moment F.S.

$b_{F.S.} = 2.0\ m$

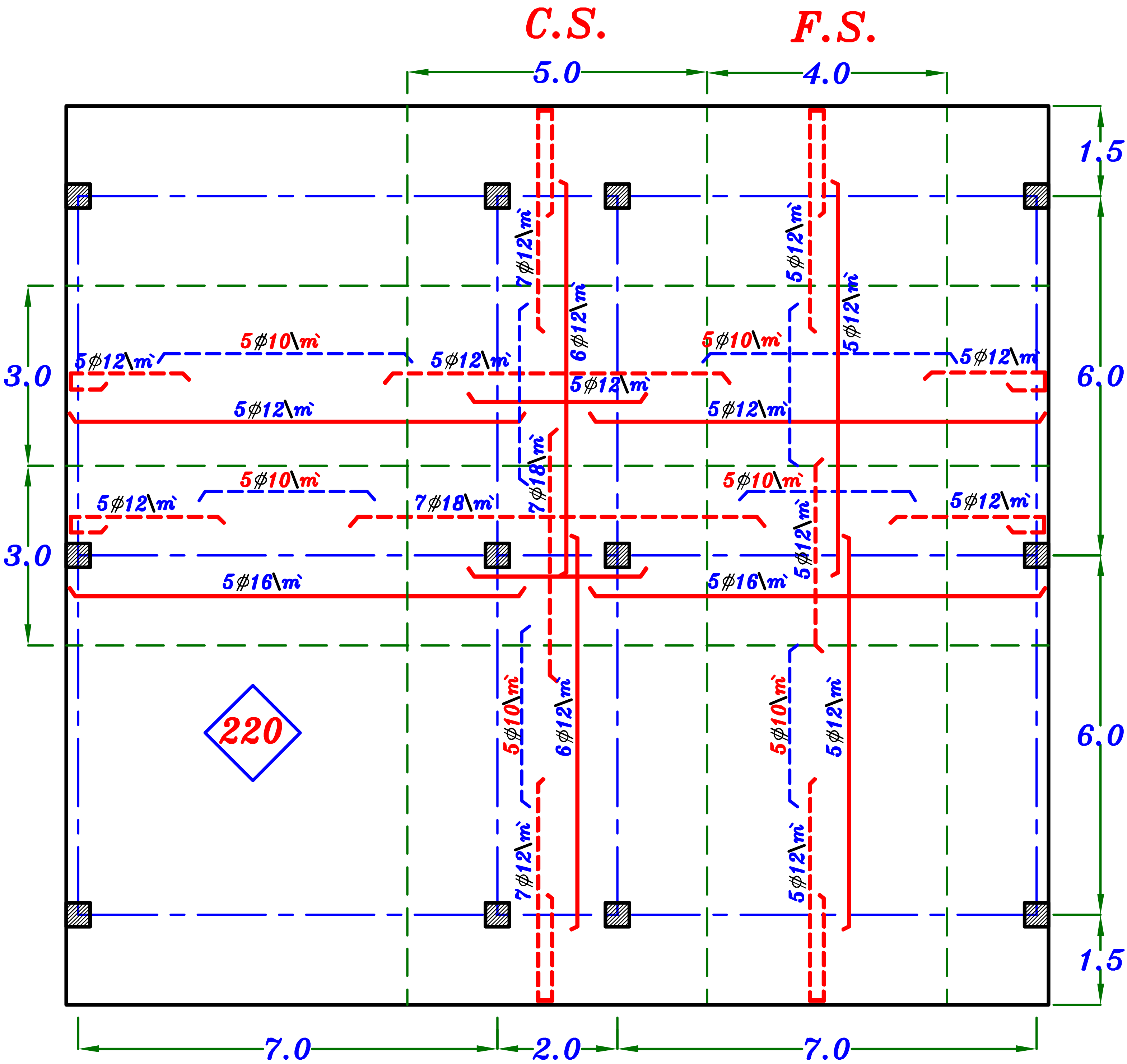


Design of sections.

$$d = t_s - 40\ mm = 220 - 40 = 180\ mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	208.87	2500	180	3.11	0.752	4286	1714	7ϕ18\m
	2	80.53	2500	180	5.01	0.826	1504	601.8	6ϕ12\m
	3	91.0	2500	180	4.71	0.824	1704	681	7ϕ12\m
Field Strip	4	53.79	2000	180	5.48	0.826	1005	502.5	5ϕ12\m
	5	59.78	2000	180	5.20	0.826	1117	558	5ϕ12\m
	6	36.0	2000	180	6.71	0.826	672	336	5ϕ12\m

Details of RFT.



Example.

The Figure shows a structural plan of a Footbridge with overall dimensions of 8.0 * 18.0 m
The slab thickness is 300 mm and provided with 400 mm thick drop panels at the intermediate columns.

Data.

$F_{cu} = 40\ N\backslash mm^2$, $F_y = 400\ N\backslash mm^2$

$F.C. = 3.0\ kN\backslash m^2$, $L.L. = 5.0\ kN\backslash m^2$

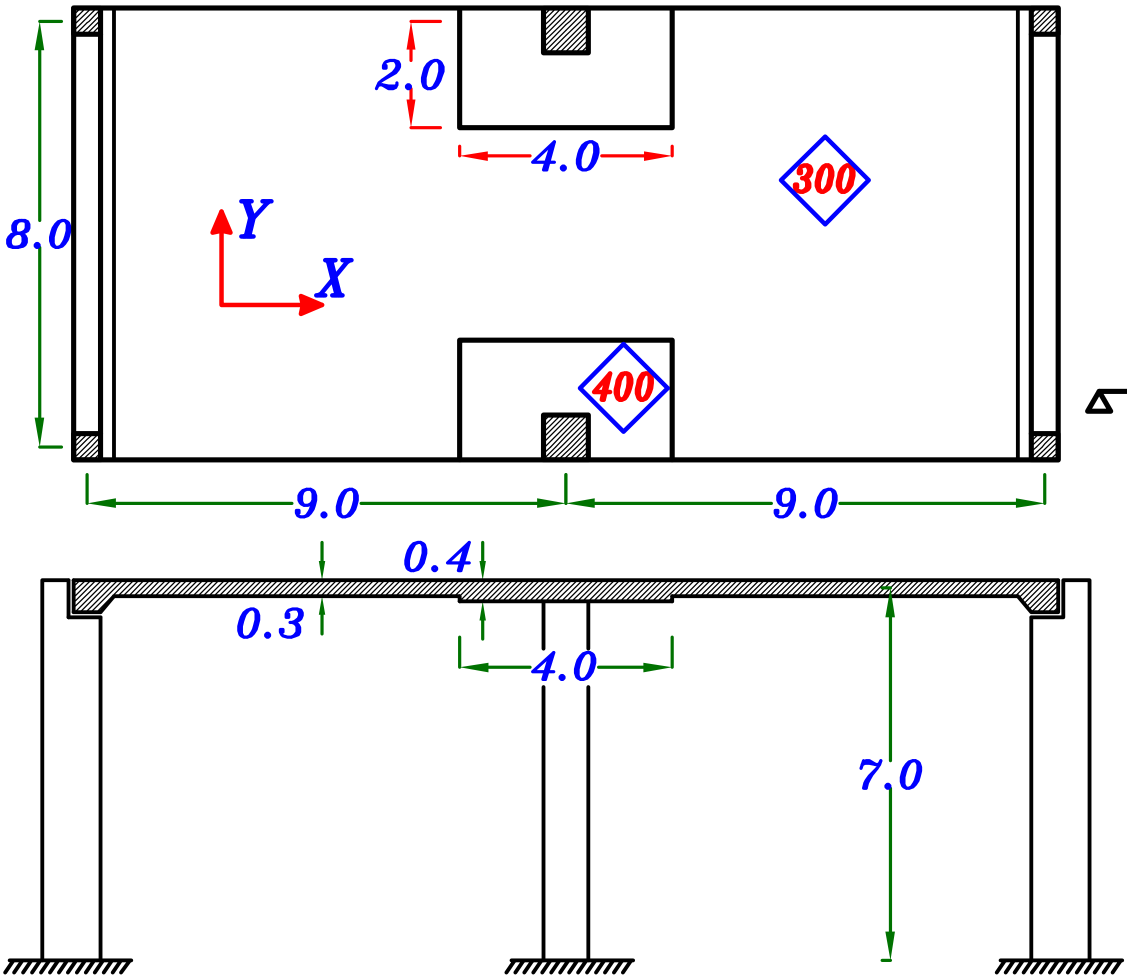
Req.

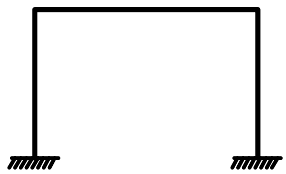
① Calculate the minimum column dimensions to satisfy the punching requirements.

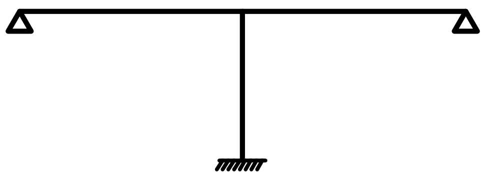
② Calculate the internal Forces (B.M. & N.F.) of the equivalent Frames in X and Y directions. (Case of total load only is required) ignoring the effect of drop panel on the slab stiffness.

③ Design the critical sections of the slab in both directions.

④ Draw a part plan showing details of reinforcement in both directions.







Solution.

1- Concrete Dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \frac{H}{15} = \frac{7000}{15} = 466.6\ mm \\ \frac{L_1}{20} = \frac{9000}{20} = 450\ mm \end{cases}$$

$$\boxed{b_{Col.} = 500\ mm}$$

$(500 * 500)$

Slab Thikness. $t_s = 300\ mm$ as given in data

Drop Panel.

$t_d = 400 - 300 = 100\ mm$ as given in data

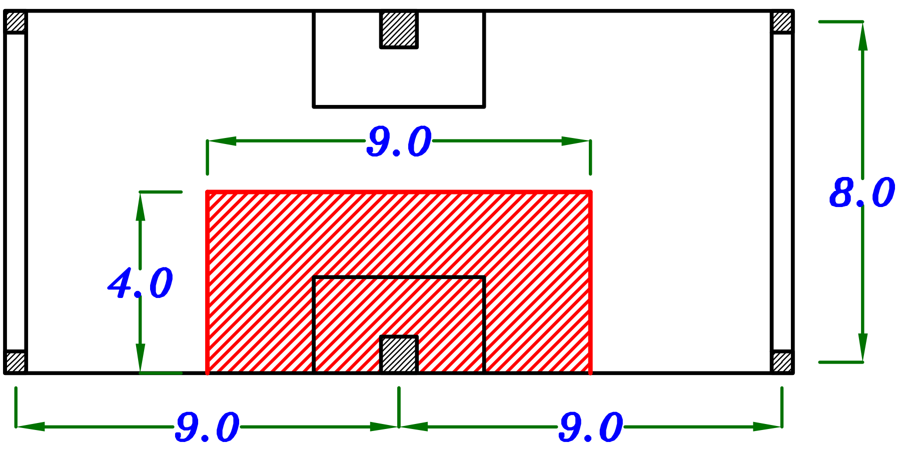
2- Loads on the Slab.

$$w_s = 1.4\left[\left(t_s + \frac{t_d}{4}\right)\gamma_c + F.C.\right] + 1.6(L.L.)$$

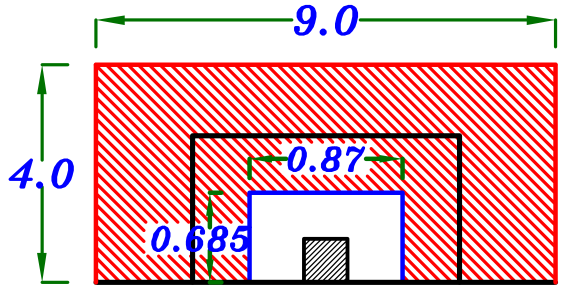
$$w_s = 1.4\left[\left(0.30 + \frac{0.1}{4}\right) * 25 + 3.0\right] + 1.6\,(5.0) = 23.57\ kN\backslash m^2$$

Check Punching For the columns.

كل عمود يحمل مساحه
من $C.L.$ البلاطه
الى $C.L.$ البلاطه الاخرى



Edge Column.



$$d = t_s - 30\,mm$$

$$= 400 - 30 = 370\ mm\ = 0.37$$

$$C + d = 0.50 + 0.37 = 0.87\ m$$

$$C + \frac{d}{2} = 0.50 + \frac{0.37}{2} = 0.685\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+\frac{d}{2})\right]$$

$$Q_{pu} = 23.57\left[9.0 * 4.0 - 0.87 * 0.685\right] = 834.47\ kN$$

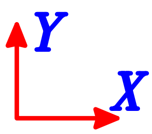
$$A_p = (b_o * d) = (2 * 685 + 870) * 370 = 828800\ \ mm^2$$

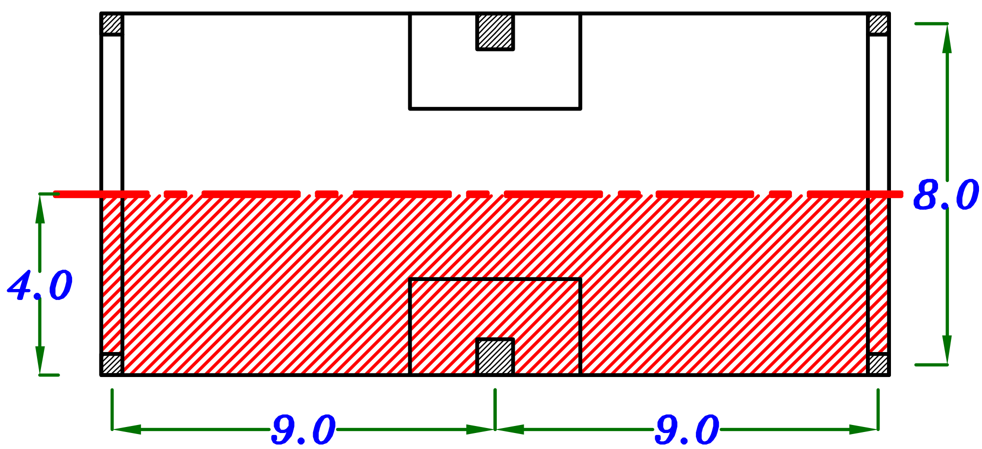
$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{834.47 * 10^3}{828800} * 1.30 = 1.31\ \ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{40}{1.5}} = 1.63\ \ N/mm^2$$

$$\boxed{q_{pu} < q_{pcu}} \longrightarrow \text{Safe Punching.}$$

X-Direction.





$$Span = L_1 = 9.0\ m$$

$$Width = L_2 = 4.0\ m$$

$$w = w_s * L_2 = 23.57 * 4.0 = 94.28\ kN/m$$

هذه الحاله لا تحتاج للحل

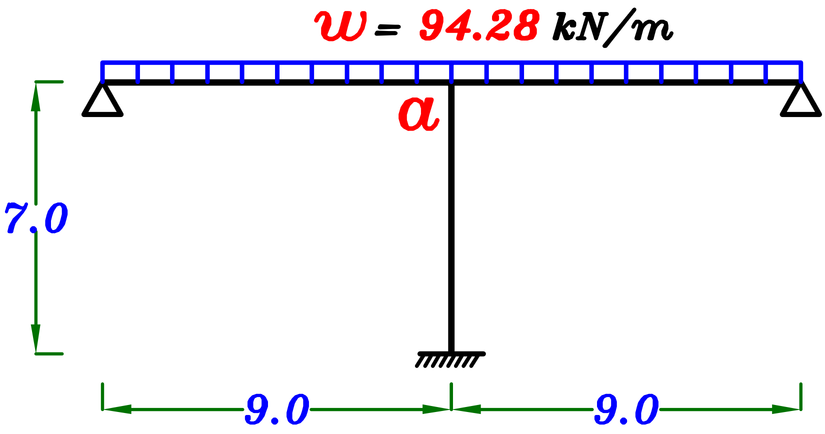
بال moment distribution

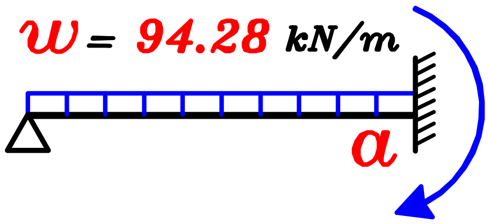
لانها symmetric

فلا يوجد عزم على العمود

فيكون العزم عند ال joint a

هو نفسه قيمه ال Fixed End Moment



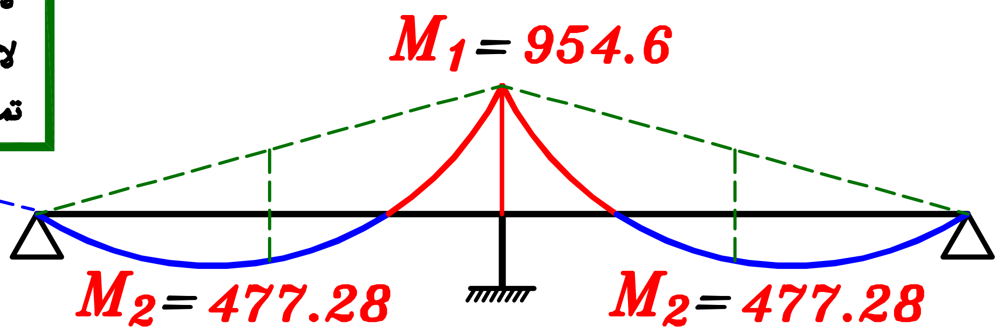


$$F.E.M. = 1.5 * \frac{w L^2}{12} = 1.5 * \frac{94.28 * 9.0^2}{12}$$

$$= 954.6 \ kN.m$$

لا يوجد عزم عند هذا ال Support
لانه hinged و البلاطه هنا مفصوله
تماما عن العمود فلا يوجد عزم بينهما

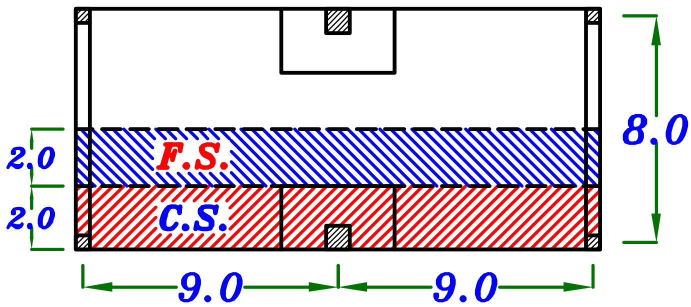
M_{total}



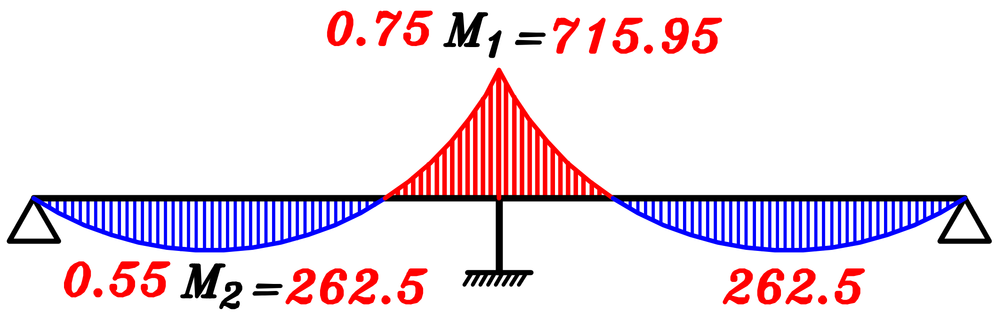
عرض ال C.S. يساوى عرض ال F.S.

$C.S. = F.S. = 2.0 \, m$

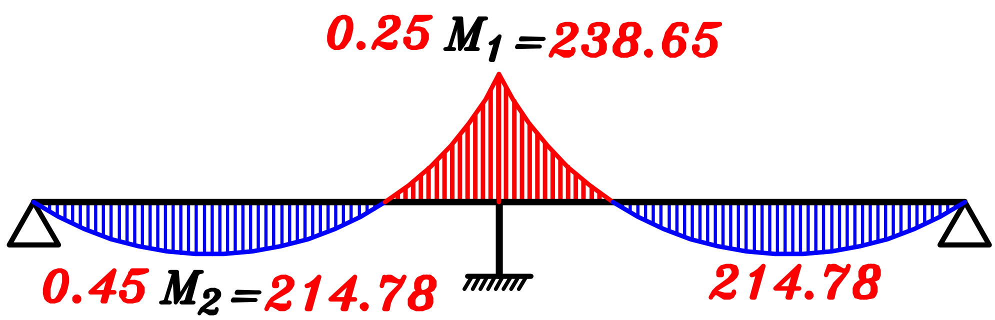
فلا يوجد Modification Factor



moment C.S.



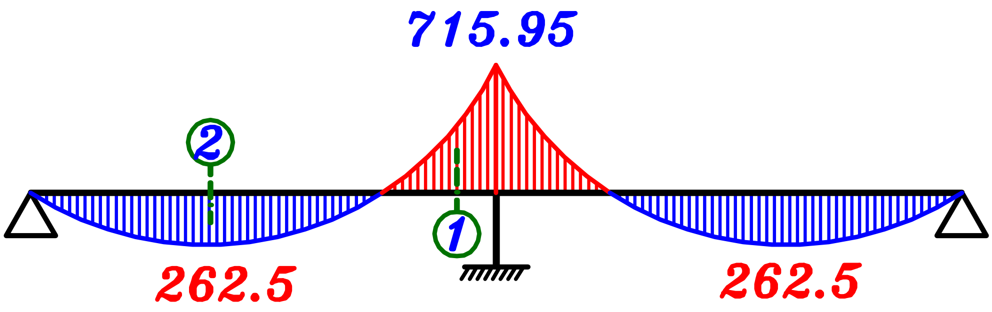
moment F.S.



6- Design the sections of the slab.

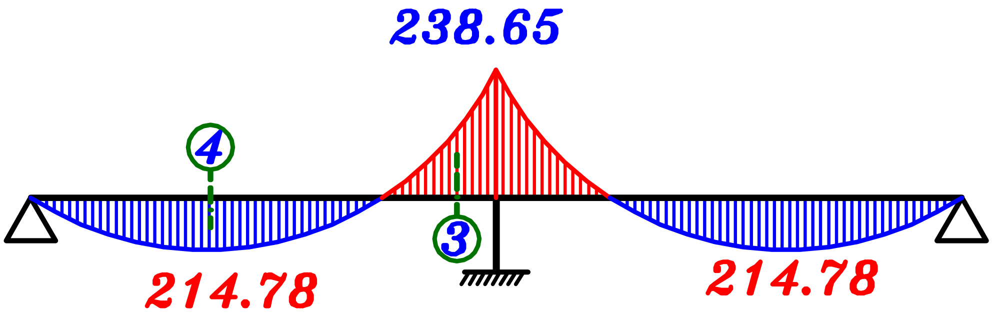
moment C.S.

$b_{F.S.} = 2.0\ m$



moment F.S.

$b_{F.S.} = 2.0\ m$

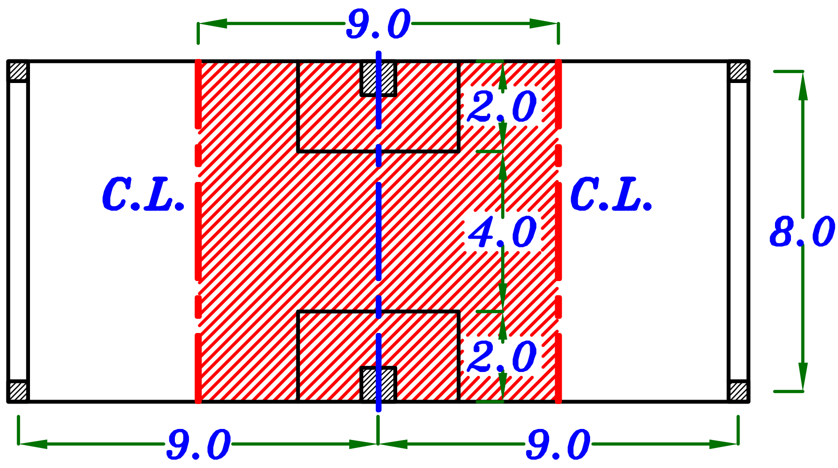


Design of sections.

$d = t_s - 30\ mm$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm^2/b)	A_s (mm^2/m)	No. of bars/m
Column Strip	1	715.95	2000	370	3.91	0.80	6046	3023	8ɸ22\m
	2	262.5	2000	270	4.71	0.822	2956	1478	6ɸ18\m
Field Strip	3	238.65	2000	270	4.94	0.826	2675	1337	6ɸ18\m
	4	214.78	2000	270	5.21	0.826	2407	1203	5ɸ18\m

Y–Direction.

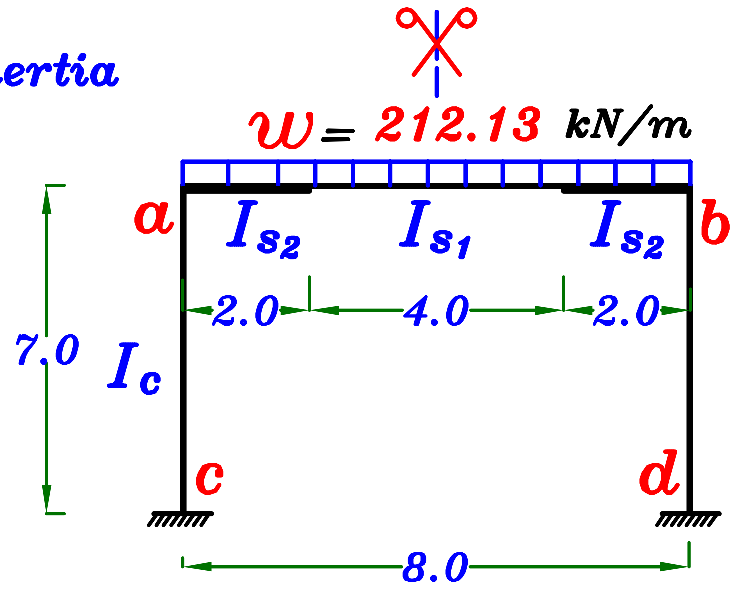


Span $= L_1 = 8.0\ m$

Width $= L_2 = 9.0\ m$

$$w = w_s * L_2 = 23.57 * 9.0 = 212.13\ kN/m$$

ⓐ Calculate Moment of Inertia For Slabs & Columns.



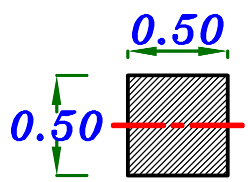
$$I_{s_1} = \frac{L_2 * t_s^3}{12} = \frac{9.0 * (0.30)^3}{12} = 20.25 * 10^{-3}\ m^4$$

$$I_{s_2} = \frac{L_2 * (t_s + \frac{t_d}{4})^3}{12} = \frac{9.0 * (0.30 + \frac{0.1}{4})^3}{12} = 25.74 * 10^{-3}\ m^4$$

$$I_{s\,av} = \frac{I_{s_1}(4.0) + I_{s_2}(2.0+2.0)}{8.0} = 22.99 * 10^{-3}\ m^4$$

عمود خارجى

$$I_c = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.50 * 0.50^3}{12} = 3.125 * 10^{-3}\ m^4$$



ⓑ Calculate the stiffness For each member.

For Slabs.
$$K_s = \frac{1}{2} * \frac{I_{s\,av}}{L} = \frac{1}{2} * \frac{22.99 * 10^{-3}}{8.0} = 1.43 * 10^{-3}$$

For Columns.
$$K_c = \frac{I_c}{h} = \frac{3.125 * 10^{-3}}{7.0} = 4.46 * 10^{-4}$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

$$\Sigma K = K_s + K_c = 1.43 * 10^{-3} + 4.46 * 10^{-4} = 1.87 * 10^{-3}$$

$$D.F.(ab) = \frac{1.43 * 10^{-3}}{1.87 * 10^{-3}} = 0.76$$

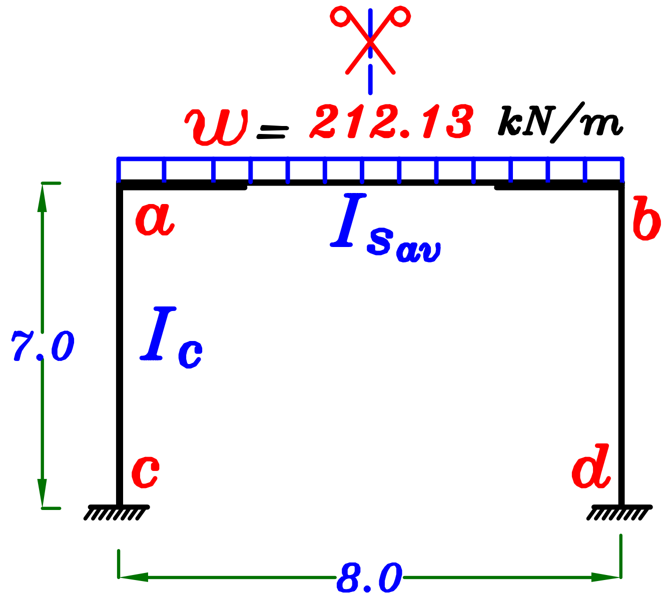
$$D.F.(ac) = \frac{4.46 * 10^{-4}}{1.87 * 10^{-3}} = 0.24$$

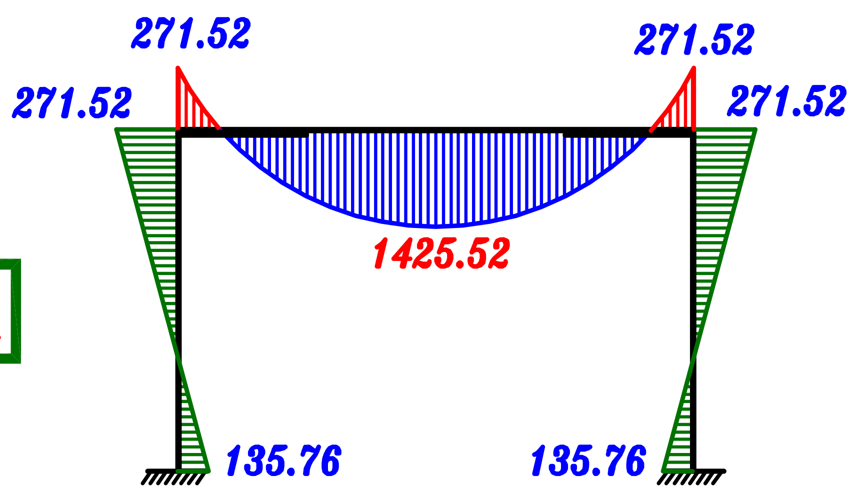
ⓓ Calculate Fixed End Moment For the Slab.

$$F.E.M.(ab) = -\frac{wL^2}{12} = -\frac{212.13 * 8.0^2}{12} = -1131.36 \text{ kN.m.}$$

$$F.E.M.(ba) = +\frac{wL^2}{12} = +\frac{212.13 * 8.0^2}{12} = +1131.36 \text{ kN.m.}$$

Joint	c	a	
member	c−a	a−c	a−b
D.F.	0	0.24	0.76
F.E.M.	0	0	−1131.36
B.M.	0	+271.52	+859.83
C.O.M.	+135.76	0	0
B.M.	0	0	0
M_F	+135.76	+271.52	−271.52

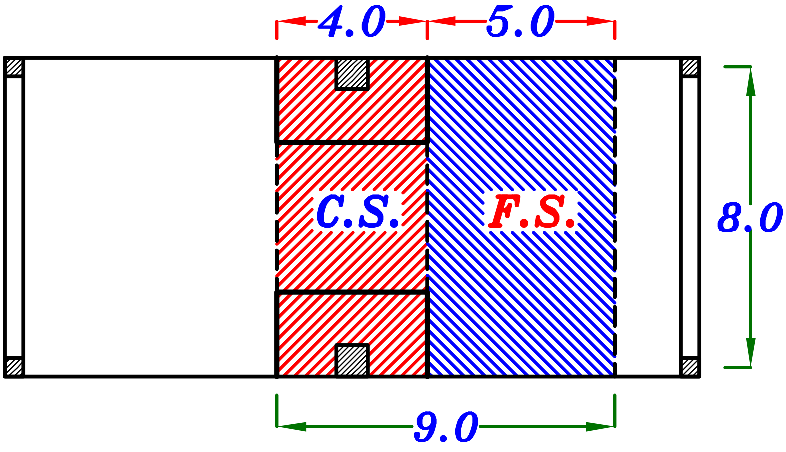




M_{total}

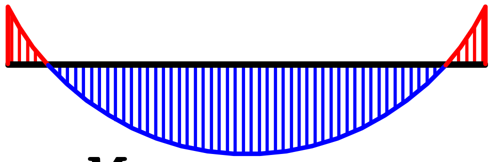
$$b_{C.S.} = \frac{L_2}{2} = 4.0\ m$$

$$b_{F.S.} = L_1 - \frac{L_2}{2} = 5.0\ m$$



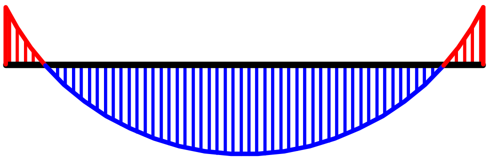
M_{total}

$M_1 = 271.52$ 271.52



$M_2 = 1425.52$

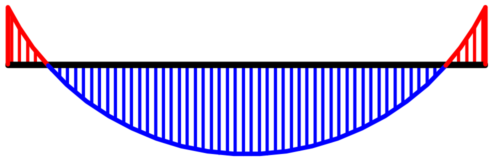
$0.20\ M_1 = 54.31$ 54.31



$0.45\ M_1 = 641.49$

$M_{F.S.}$

$0.80\ M_1 = 217.21$ 217.21



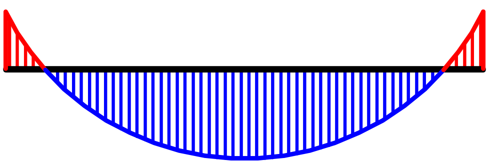
$0.55\ M_1 = 784.03$

$M_{C.S.}$

$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{5.0}{4.5}$$

$$(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}$$

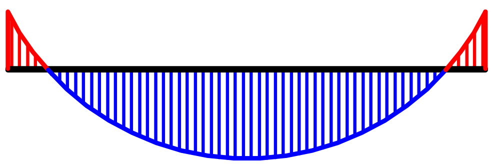
$54.31 * \frac{5.0}{4.5} = 60.34$ 60.34



$641.49 * \frac{5.0}{4.5} = 712.76$

$M_{F.S.\,mod.}$

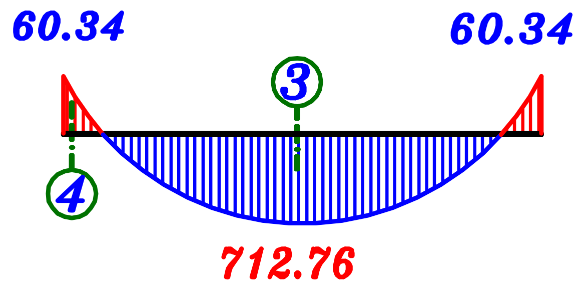
$217.21 + 54.31 - 60.34$
$= 211.18$ 211.18

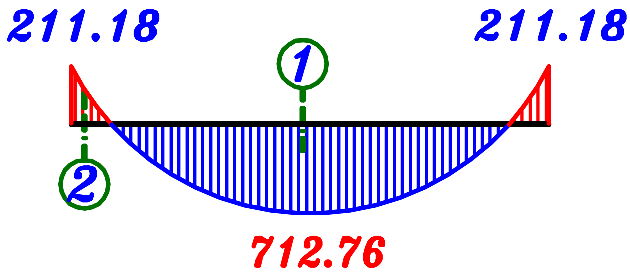


$784.03 + 641.49 - 712.76 = 712.76$

$M_{C.S.\,mod.}$

Design the sections of the slab.





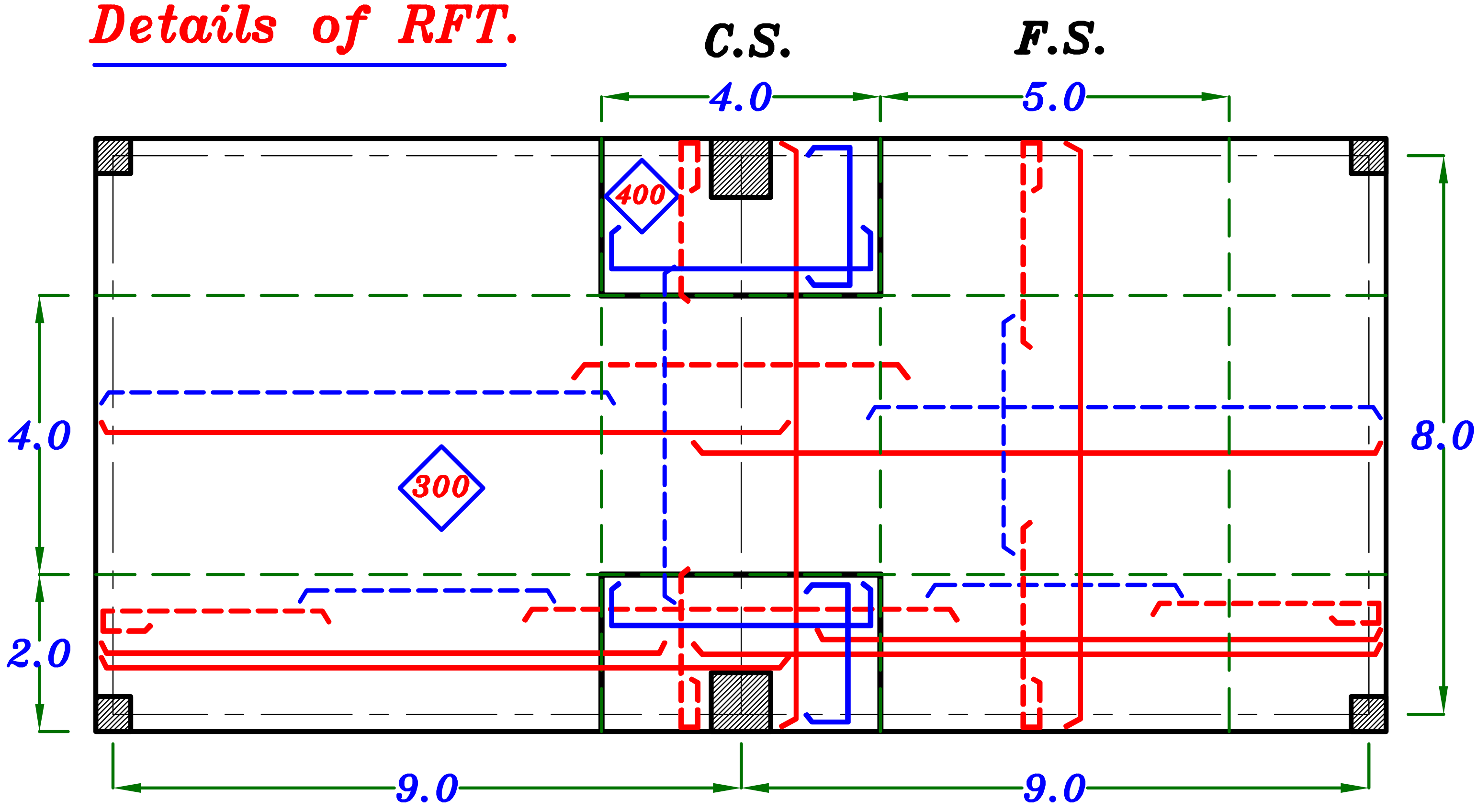
$M_{F.S.mod.}$ $b_{F.S.} = 4.0$ m $M_{C.S.mod.}$ $b_{C.S.} = 5.0$ m

Design of sections.

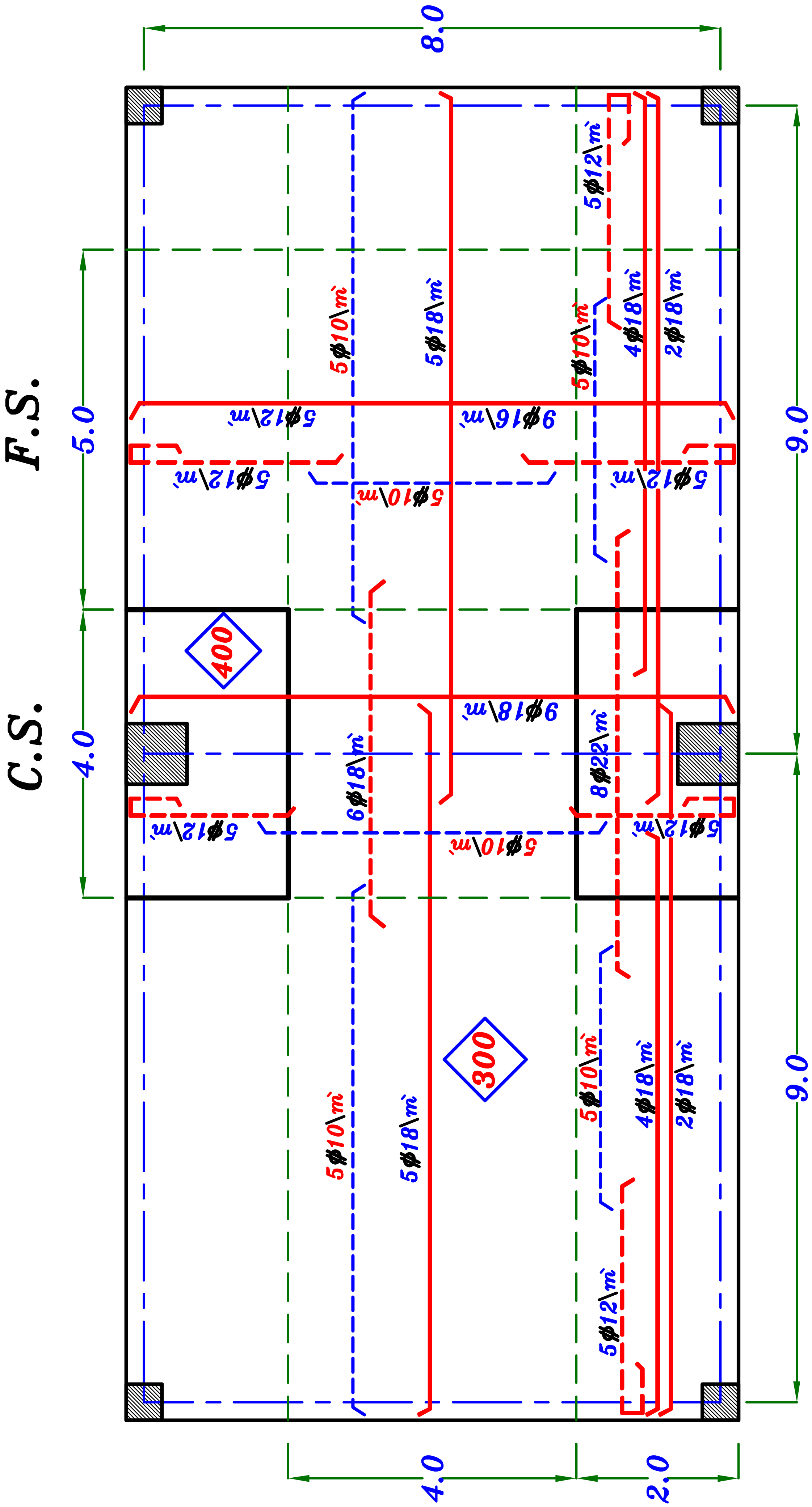
$d = t_s - 40$ mm

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	712.76	4000	260	12.31	0.826	8297	2074	9ϕ18\m
	2	211.18	4000	360	31.3	0.806	1775	443	5ϕ12\m
Field Strip	3	712.76	5000	260	13.7	0.826	8297	1659	9ϕ16\m
	4	60.34	5000	260	47.3	0.826	702.4	140	5ϕ12\m

Details of RFT.



Details of RFT.



Example.

The given plan shows a plan and cross section of Flat Slab Shed.

Data. $F_{cu} = 25\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

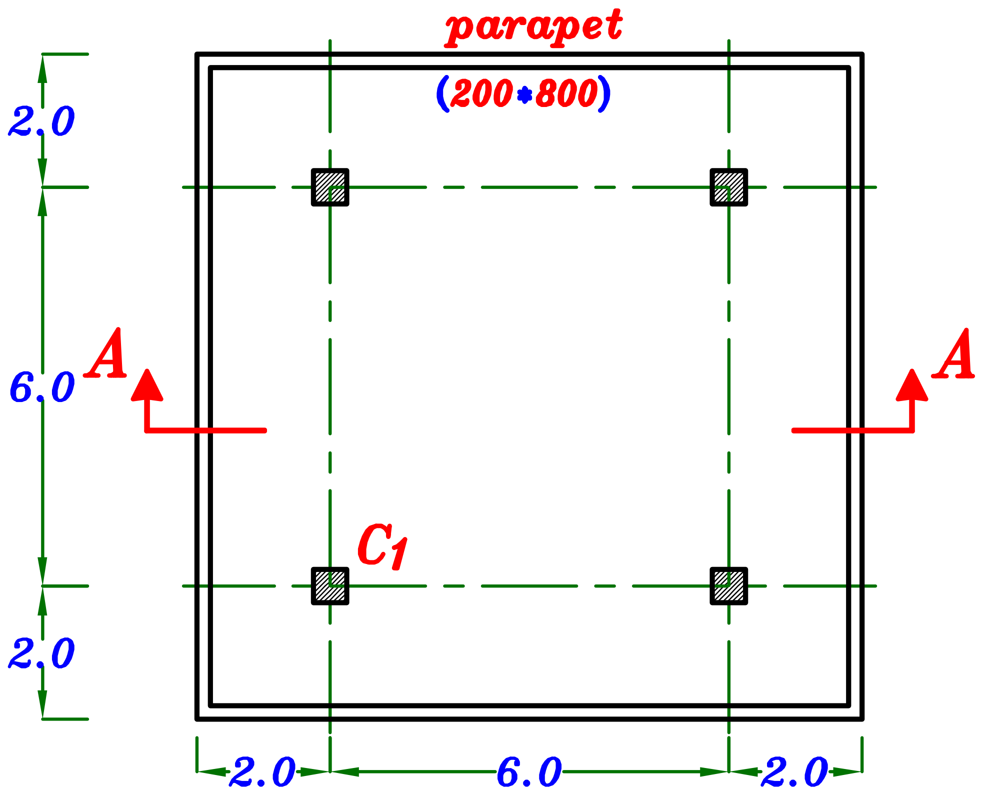
$F.C. = 2.5\ kN\backslash m^2$, $L.L. = 1.5\ kN\backslash m^2$

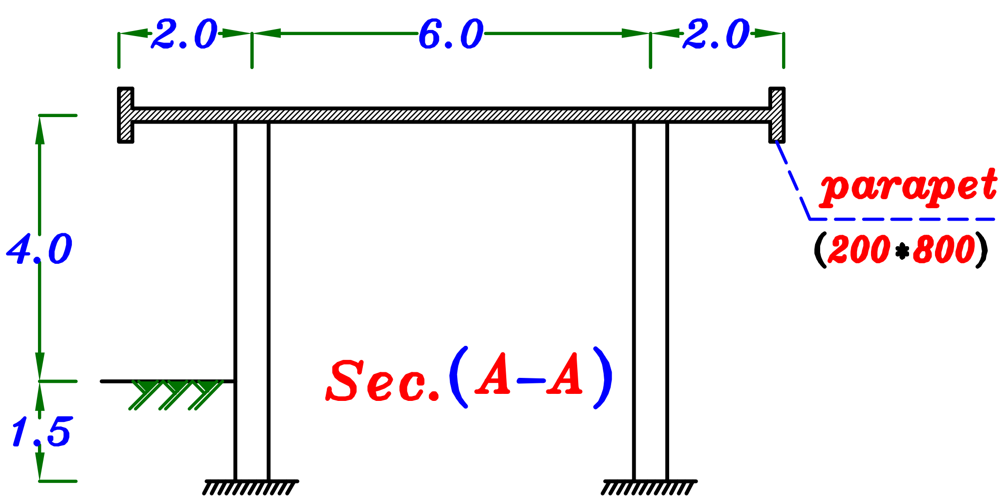
Req.

① Check punching of column C_1

② Complete design For the slab.

③ Draw details of reinforcement in plan.





Solution.

1- Concrete Dimensions.

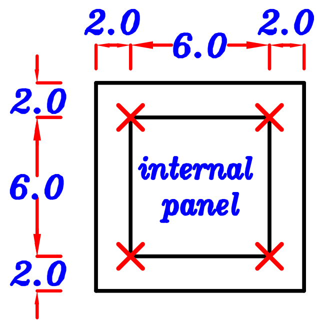
Column dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{5500}{15} = 366.6\ mm \\ \dfrac{L_1}{20} = \dfrac{6000}{20} = 300\ mm \end{cases}$$

$$\boxed{b_{Col.} = 400\ mm}$$

$(400 * 400)$

Slab Thikness.



الباكيه تعتبر داخليه نظرا لوجود Cantilever

$L_1 = 6.0\ m$

Internal panel $\quad t_s = \dfrac{L_1}{36} = \dfrac{6000}{36} = 166.6\ mm$

Cantilever $\quad t_s = \dfrac{L_c}{10} = \dfrac{2000}{10} = 200\ mm$

$$\boxed{t_s = 200\ mm}$$

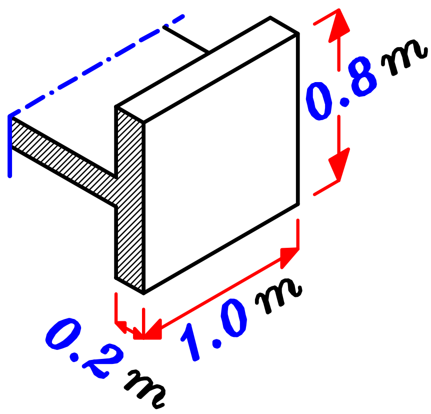
2- Loads on the Slab.

$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)$$

$$w_s = 1.4\,(0.20 * 25 + 2.50) + 1.6\,(1.50) = 12.90\ kN\backslash m^2$$

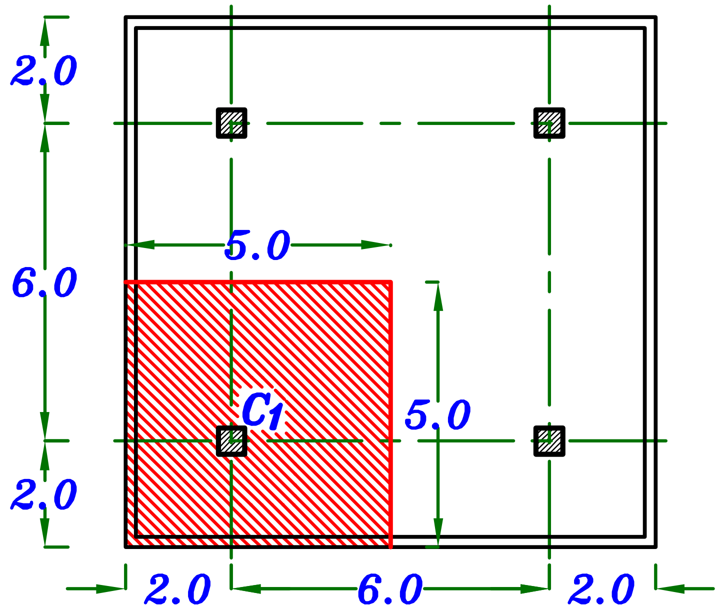
3- parapet weight.

وزن متر طولى للدروه

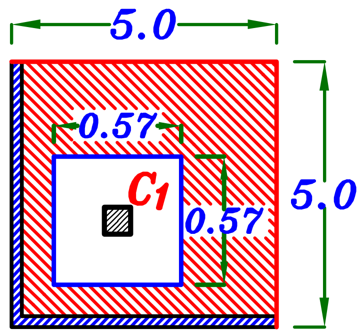


$$P = 1.4\,(0.2 * 0.8 * 1.0)\ 25 = 5.60\ kN/m$$

Check Punching of interior column C_1



$\underline{\underline{C_1}}$ Interior Column.



$$d = t_s - 30\,mm = 200 - 30 = 170\ mm = 0.17\ m$$

$$C+d = 0.40 + 0.17 = 0.57\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+d)\right] + \text{parapet weight}$$

طول الدروه

$$Q_{pu} = 12.90\left[5.0 * 5.0 - 0.57 * 0.57\right] + 5.60 * (5.0 + 5.0)$$

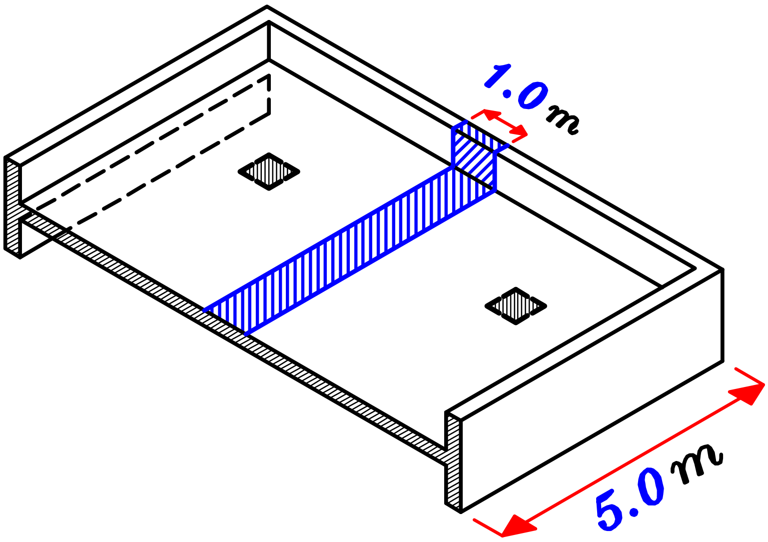
$$= 374.31\ kN$$

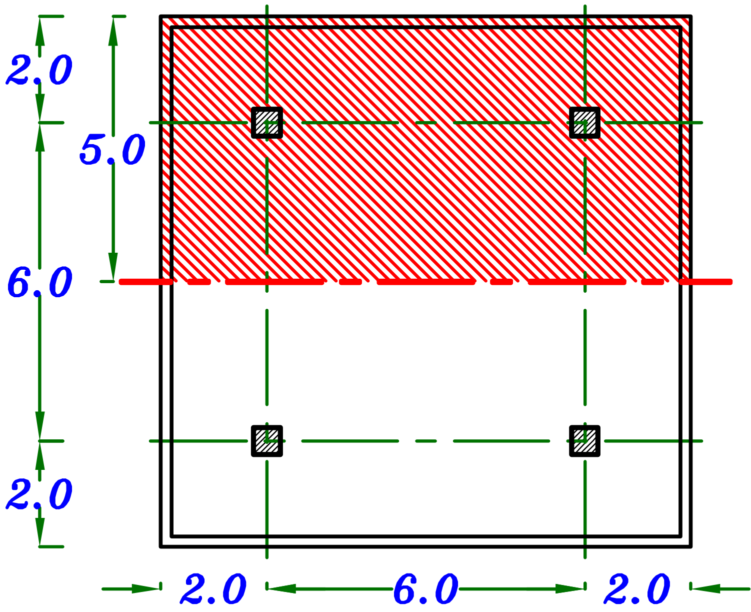
$$A_p = (b_o * d) = (4 * 570) * 170 = 387600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{374.31 * 10^3}{387600} * 1.15 = 1.11\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{25}{1.5}} = 1.29\ N/mm^2$$

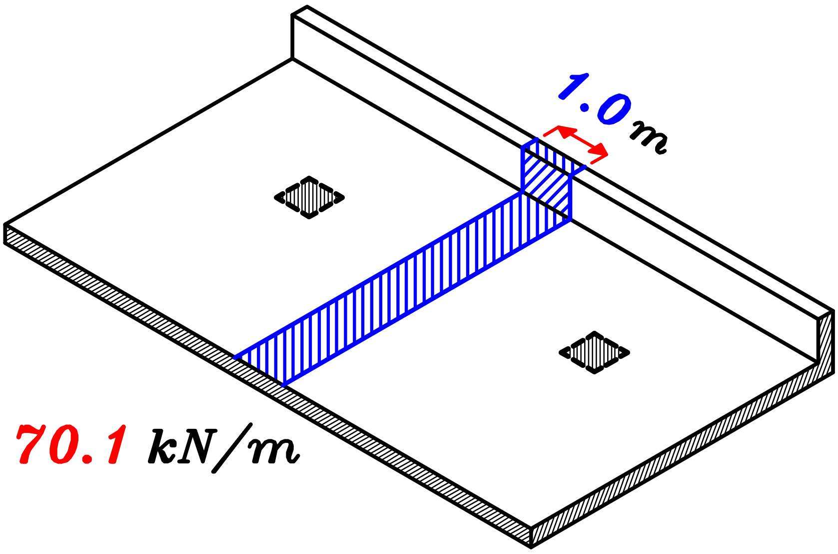
$\boxed{q_{pu} < q_{pcu}}$ ⟶ Safe Punching.





$Span = L_1 = 6.0\ m$

$Width = L_2 = 5.0\ m$

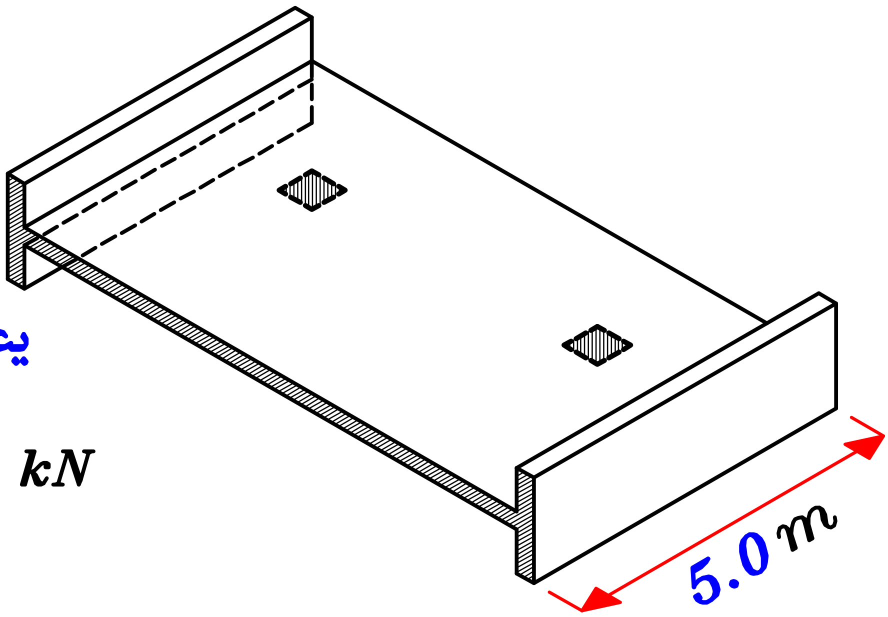


وزن الدروه فى هذا الاتجاه

يعتبر distributed load

$w = w_s * L_2 + paraprt$

$w = 12.90 * 5.0 + 5.60 = 70.1\ kN/m$

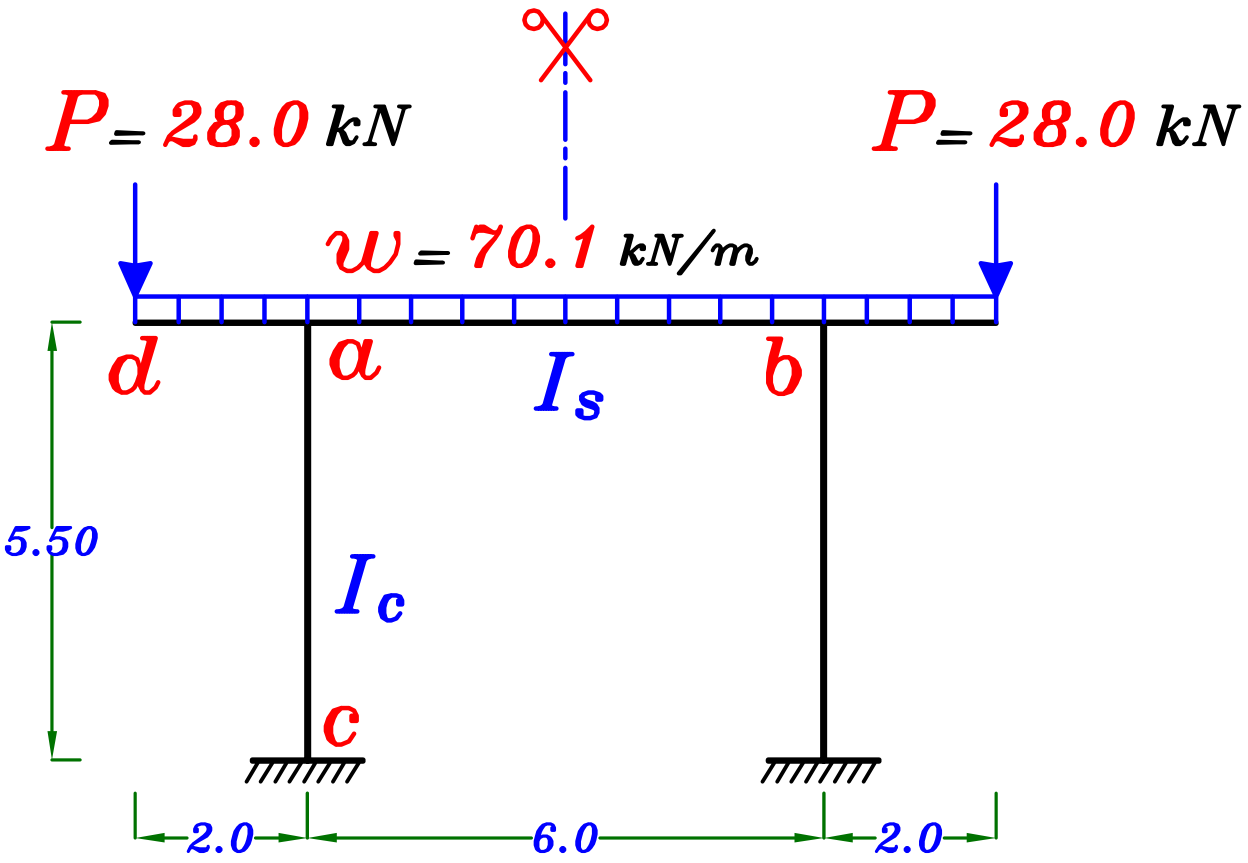


وزن الدروه فى هذا الاتجاه

يعتبر Consentrated load

$P = 5.60 * 5.0 = 28.0\ kN$

Use Moment Distribution.



ⓐ **Calculate Moment of Inertia For Slabs & Columns.**

عمود خارجي

$$I_c = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.40 * 0.40^3}{12} = 1.28 * 10^{-3}\ m^4$$

0.40 × 0.40

$$I_s = \frac{L_2 * t_s^3}{12} = \frac{5.0 * 0.20^3}{12} = 3.33 * 10^{-3}\ m^4$$

5.0 × 0.20

ⓑ **Calculate the stiffness For each member.**

For Slabs.

$$K_s = \frac{1}{2} * \frac{I_s}{L} = \frac{1}{2} * \frac{3.33 * 10^{-3}}{6.0} = 2.775 * 10^{-4}$$

For Columns.

$$K_c = \frac{I_c}{h} = \frac{1.28 * 10^{-3}}{5.5} = 2.32 * 10^{-4}$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

$$\Sigma K = K_s + K_c = 2.775 * 10^{-4} + 2.32 * 10^{-4} = 5.095 * 10^{-4}$$

$$D.F.(ab) = \frac{2.775 * 10^{-4}}{5.095 * 10^{-4}} = 0.544$$

$$D.F.(ac) = \frac{2.32 * 10^{-4}}{5.095 * 10^{-4}} = 0.456$$

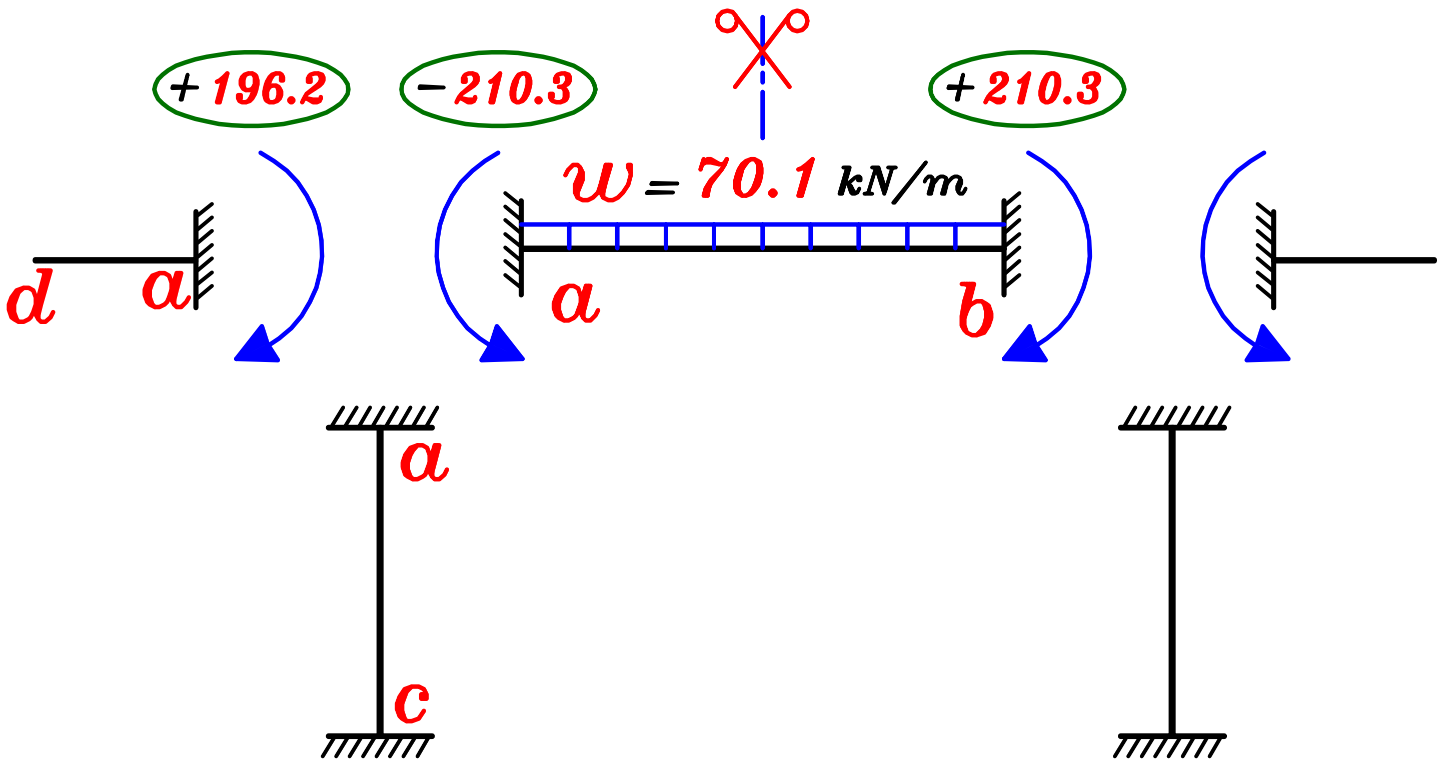
$$D.F.(ad) = Zero$$

ⓓ Calculate Fixed End Moment For the Slab.

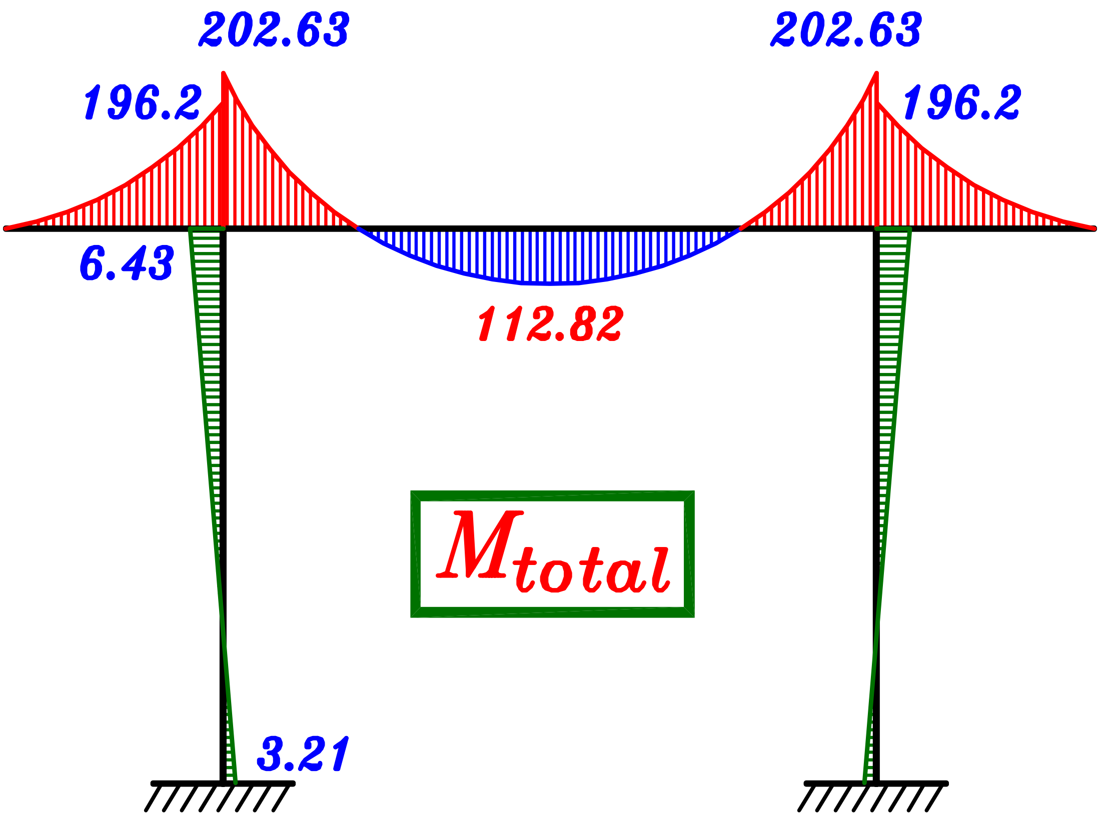
$$F.E.M.(ab) = -\frac{wL^2}{12} = -\frac{70.1 * 6.0^2}{12} = -210.3 \ kN.m.$$

$$F.E.M.(ba) = +\frac{wL^2}{12} = +\frac{70.1 * 6.0^2}{12} = +210.3 \ kN.m.$$

$$F.E.M.(ad) = +\frac{wL^2}{2} + PL = +\frac{70.1 * 2.0^2}{2} + 28.0 * 2.0 = +196.2 \ kN.m.$$



Joint	c	a		
member	c − a	a − c	a − d	a − b
D.F.	0	0.456	0	0.544
F.E.M.	0	0	+196.2	−210.3
B.M.	0	+6.43	0	+7.67
C.O.M.	+3.21	0	0	0
B.M.	0	0	0	0
M_F	+3.21	+6.43	+196.2	−202.63

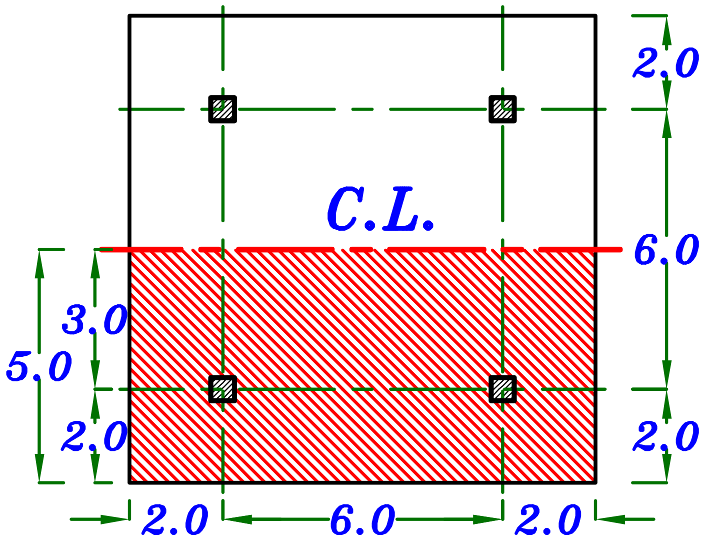


Modification Factor.

عرض شريحه التصميم الكليه

Total Strip width =

$$= \frac{6.0}{2.0} + 2.0 = 5.0\ m$$



$$\boxed{b_{C.S.} = \frac{L_2}{4} + \text{Width of the Cantilever}}$$

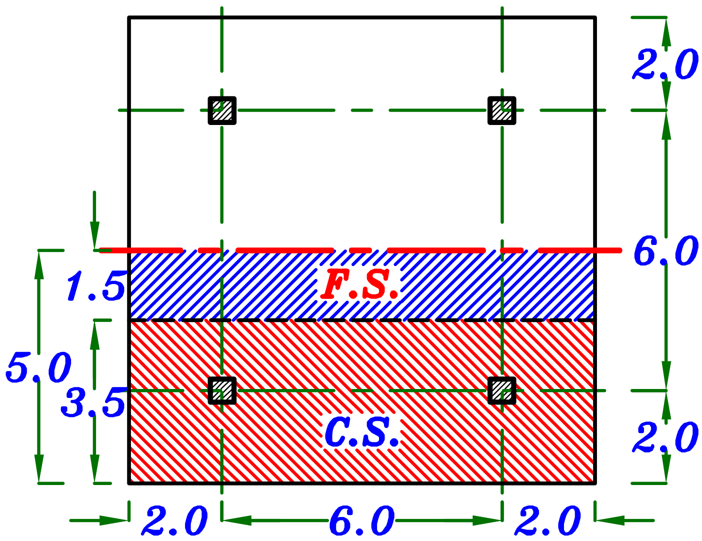
يؤخذ عرض ال Column strip

$$b_{C.S.} = \frac{6.0}{4} + 2.0 = 3.5\ m$$

$$\boxed{b_{F.S.} = \text{Total Strip width} - b_{C.S.}}$$

و يؤخذ عرض ال Field strip

$$b_{F.S.} = 5.0 - 3.5 = 1.50\ m$$

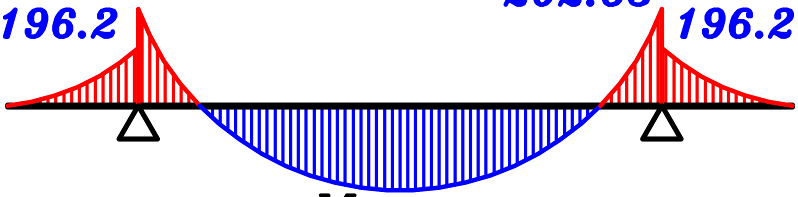


Modification Factor For Field Strip

$$\boxed{M.F. = \frac{\text{العرض الحقيقى لل Field strip}}{\frac{1}{2}\ \text{عرض الشريحه الكلى}} = \frac{1.5}{2.50} = 0.60}$$

Distribute the moment of the Frame on Column Strip and Field Strip.

$\boxed{M_{total}}$

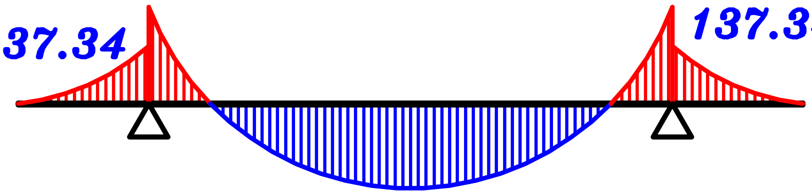


$M_1 * \frac{3.5}{5.0} = 137.74$

عرض ال C.S.

عرض الشريحه كله

moment C.S.

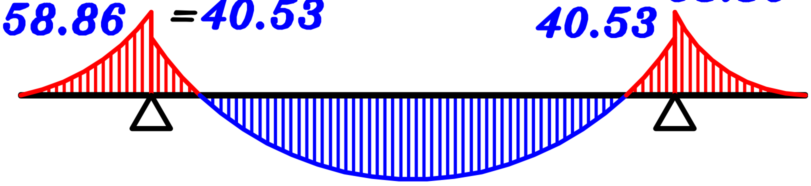


$M_1 * \frac{1.5}{5.0} = 58.86$

عرض ال F.S.

عرض الشريحه كله

moment F.S.

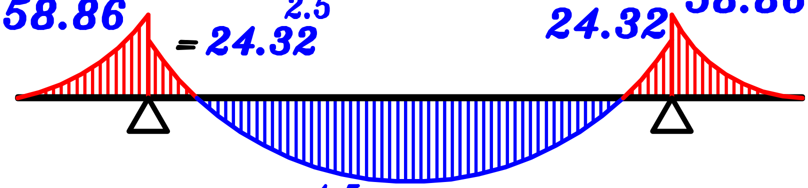


Modification Factor $= \frac{1.5}{2.5}$

$$\boxed{(M_{F.S})_{mod.} = (M_{F.S}) * \text{Modification Factor} = (M_{F.S}) * \frac{1.5}{2.5}}$$

Modified moment F.S.

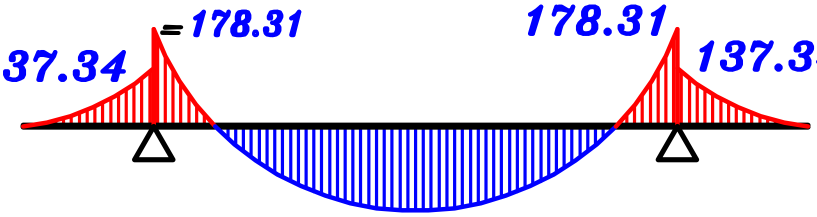
$b_{F.S.} = 1.5\ m$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

Modified moment C.S.

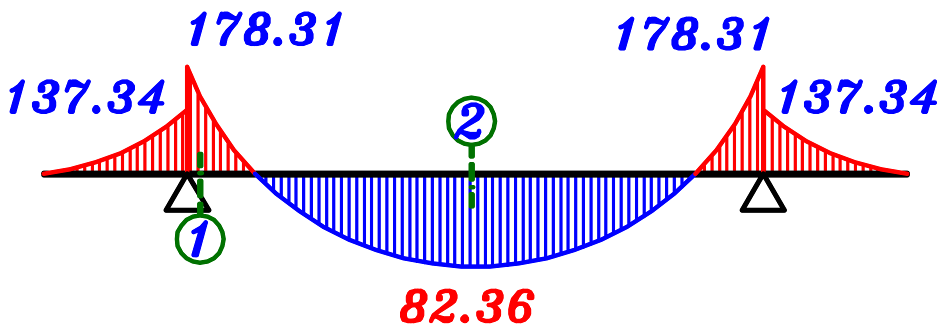
$b_{C.S.} = 3.5\ m$



Design the sections of the slab.

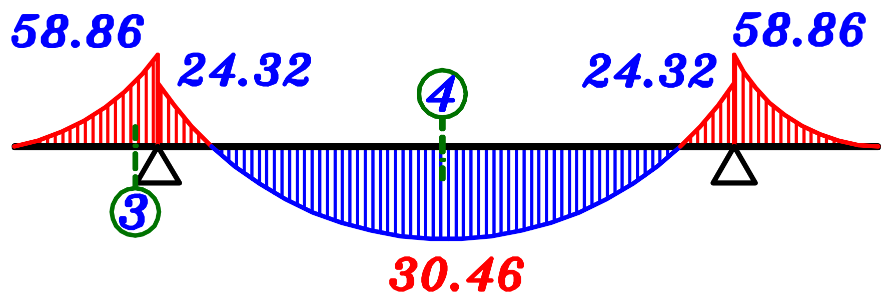
Modified moment C.S.

$b_{C.S.} = 3.5\ m$



Modified moment F.S.

$b_{F.S.} = 1.5\ m$

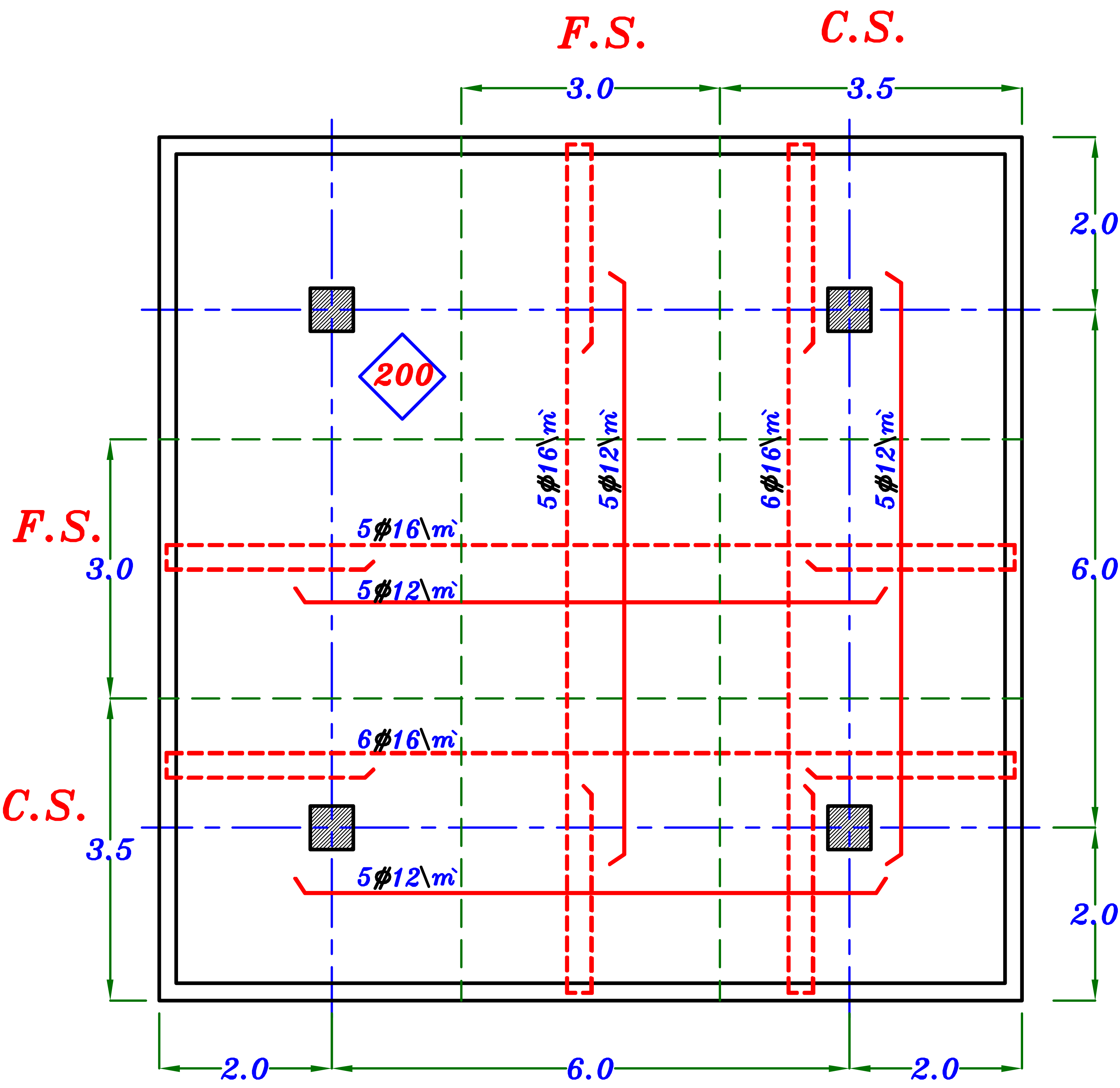


Design of sections.

$$d = t_s - 40\ mm = 200 - 40 = 160\ mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	178.31	3500	160	3.54	0.782	3958	1130	6∅16\m
	2	82.36	3500	160	5.21	0.826	1731	494	5∅12\m
Field Strip	3	58.86	1500	160	4.03	0.804	1271	847	5∅16\m
	4	30.46	1500	160	5.61	0.826	640	426	5∅12\m

Details of RFT.



Example.

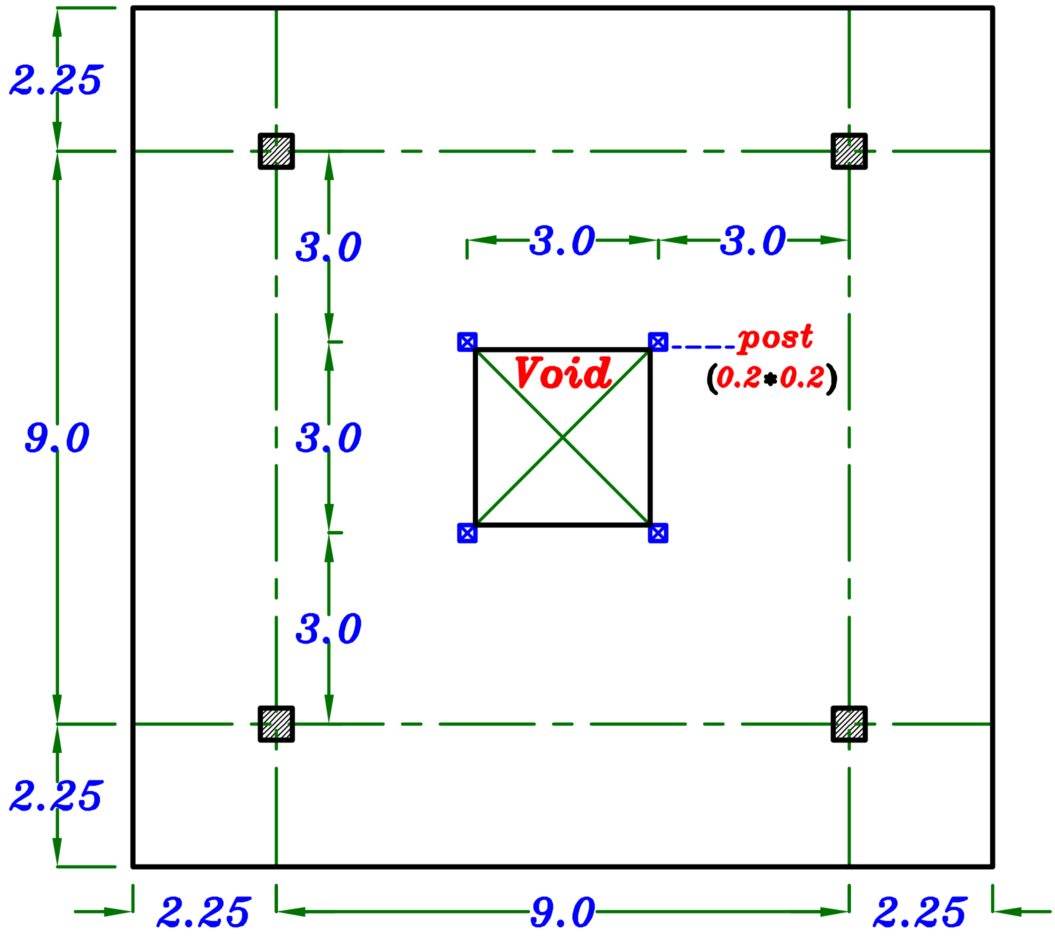
Data. $F_{cu} = 30\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

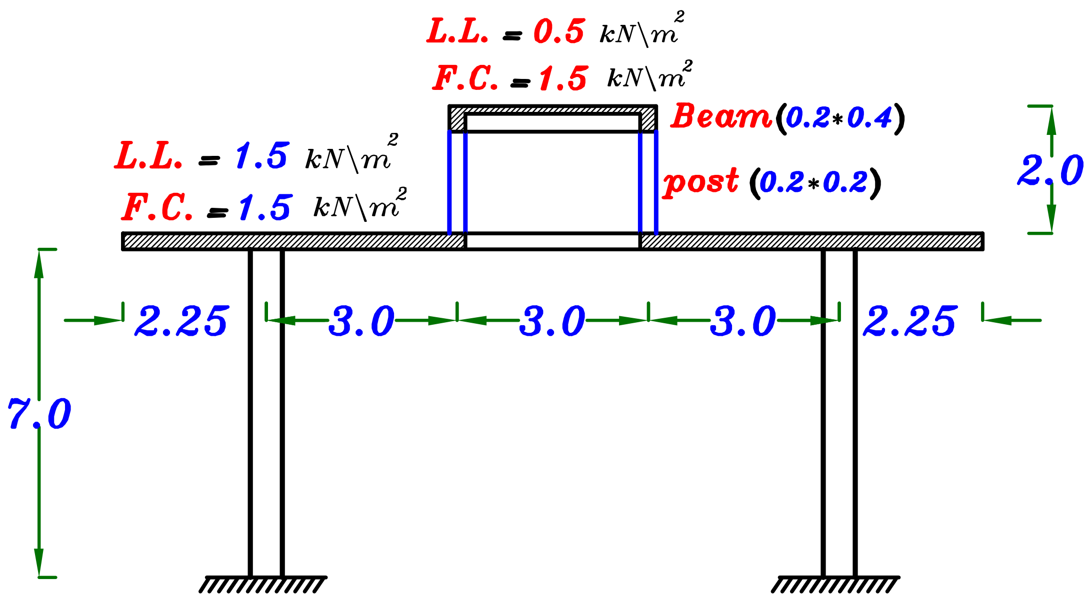
Req.

① Check punching shear of the slab at one of the columns.

② Using Frame analysis method, Design the slab assuming constant inertia and uniform load distribution (Case of total load only is required).

③ Draw a half plan showing details of reinforcement in both directions.





Solution.

1- Concrete Dimensions.

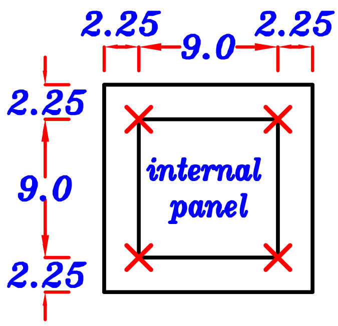
Column dimensions.

$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{7000}{15} = 466.6\ mm \\ \dfrac{L_1}{20} = \dfrac{9000}{20} = 450\ mm \end{cases}$$

$$\boxed{b_{Col.} = 500\ mm}$$

$(500 * 500)$

Slab Thikness.



Flat Slab.

$L_1 = 9.0\ m$

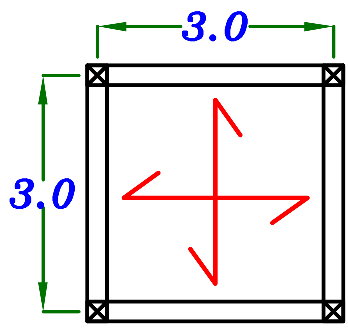
Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{9000}{36} = 250\ mm$

Cantilever $t_s = \dfrac{L_c}{10} = \dfrac{2250}{10} = 225\ mm$

$$\boxed{t_s = 250\ mm}$$

Solid slab. Two way

$t_s = \dfrac{L_s}{35} = \dfrac{3000}{35} = 85.7\ mm$ $\boxed{t_s = 100\ mm}$



2- Loads on the Slab.

Flat Slab.

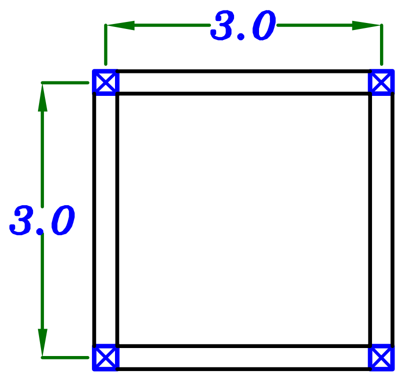
$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)$$

$$w_s = 1.4\,(0.25 * 25 + 1.50) + 1.6\,(1.50) = 13.25\ kN\backslash m^2$$

Solid Slab.

$$w_s = 1.4\,(0.10 * 25 + 1.50) + 1.6\,(0.50) = 6.40\ kN\backslash m^2$$

Loads on the Post.



Beam (200*400)

$$o.w._{(beam)} = 1.4 * 0.2 * 0.4 * 25 = 2.8 \ kN/m$$

Post (200*200)

$$o.w._{(Post)} = 1.4 * 0.2 * 0.2 * 2.0 * 25 = 2.8 \ kN$$

لتحديد الحمل على ال post الواحد

نحسب الوزن الكلى للشخشيخه من بلاطه و كمرات و posts

و نقسم الوزن الكلى على ٤ .

$$Total\ Weight = Slab + 4\ Beams + 4\ Posts$$

$$Slab = w_s * area = 6.40 * (3.0 * 3.0) = 57.6 \ kN$$

$$4\ Beams = o.w._{(beam)} * \text{طول الكمرات} = 2.8\,(3.0 * 4.0) = 33.6 \ kN$$

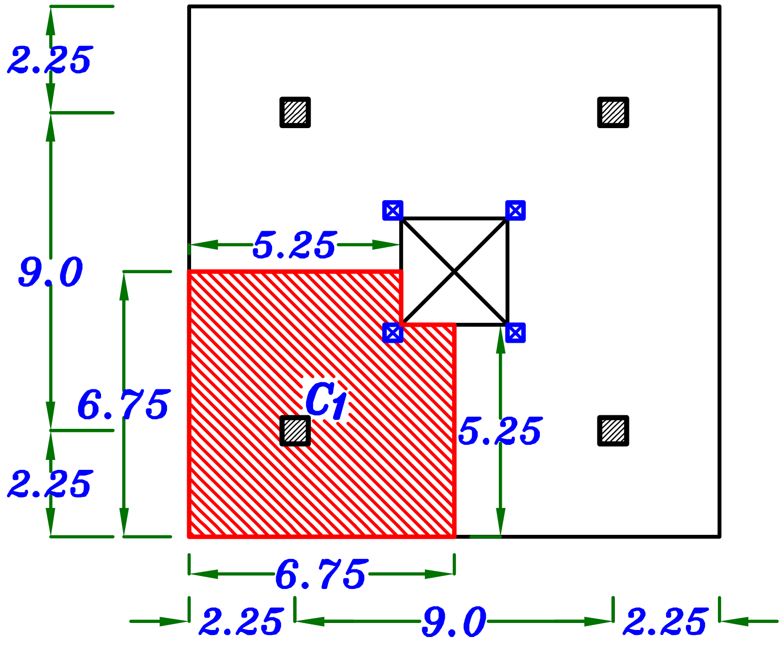
$$4\ Posts = 4 * 2.8 = 11.2 \ kN$$

$$Total\ Weight = 57.6 + 33.6 + 11.2 = 102.4 \ kN$$

$$Load\ on\ One\ Post = \frac{Total\ Weight}{4} = \frac{102.4}{4} = 25.6 \ kN$$

Check Punching on interior column C_1

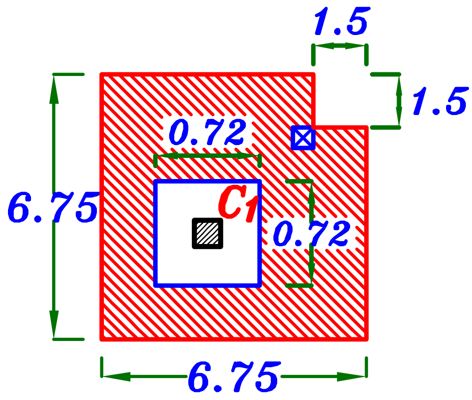
C_1 Interior Column.



$$d = t_s - 30\ mm$$

$$= 250 - 30 = 220\ mm = 0.22\ m$$

$$C + d = 0.50 + 0.22 = 0.72\ m$$



$$Q_{pu} = w_s \left[L_1 * L_2 - void - (C_1 + d)(C_2 + d) \right] + Post$$

$$Q_{pu} = 13.25 \left[6.75 * 6.75 - (1.5 * 1.5) - (0.72 * 0.72) \right] + 25.6$$

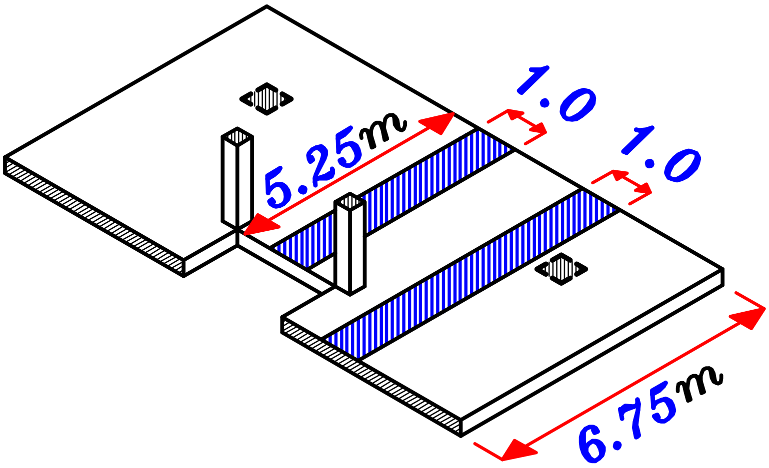
$$= 592.6\ kN$$

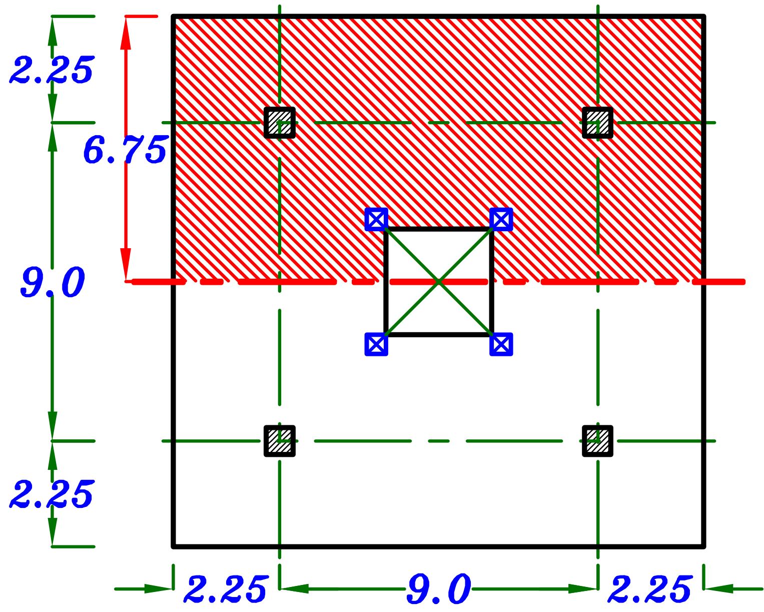
$$A_p = (b_o * d) = (4 * 720) * 220 = 633600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{592.6 * 10^3}{633600} * 1.15 = 1.07\ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$$

$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.





$$w_1 = w_s * L_2 = 13.25 * 6.75 = 89.43 \ kN/m$$

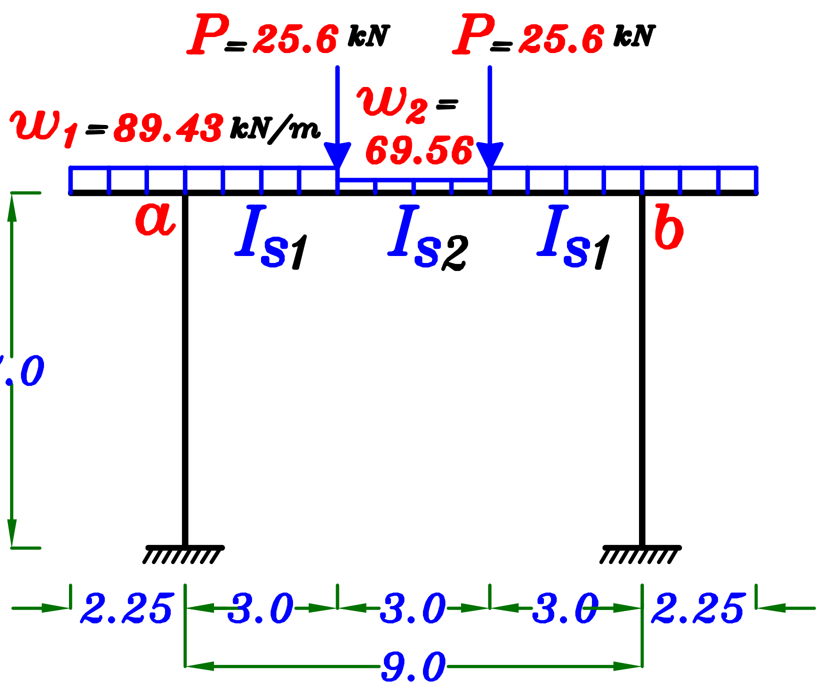
$$w_2 = w_s * L_2 = 13.25 * 5.25 = 69.56 \ kN/m$$

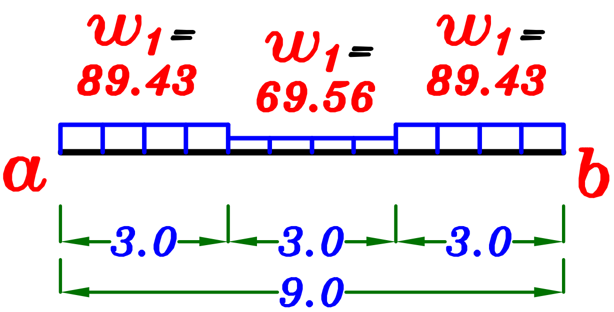
للحل بطريقة

moment distribution

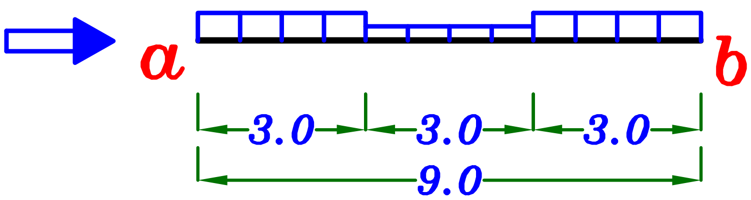
member يجب أن يكون كل

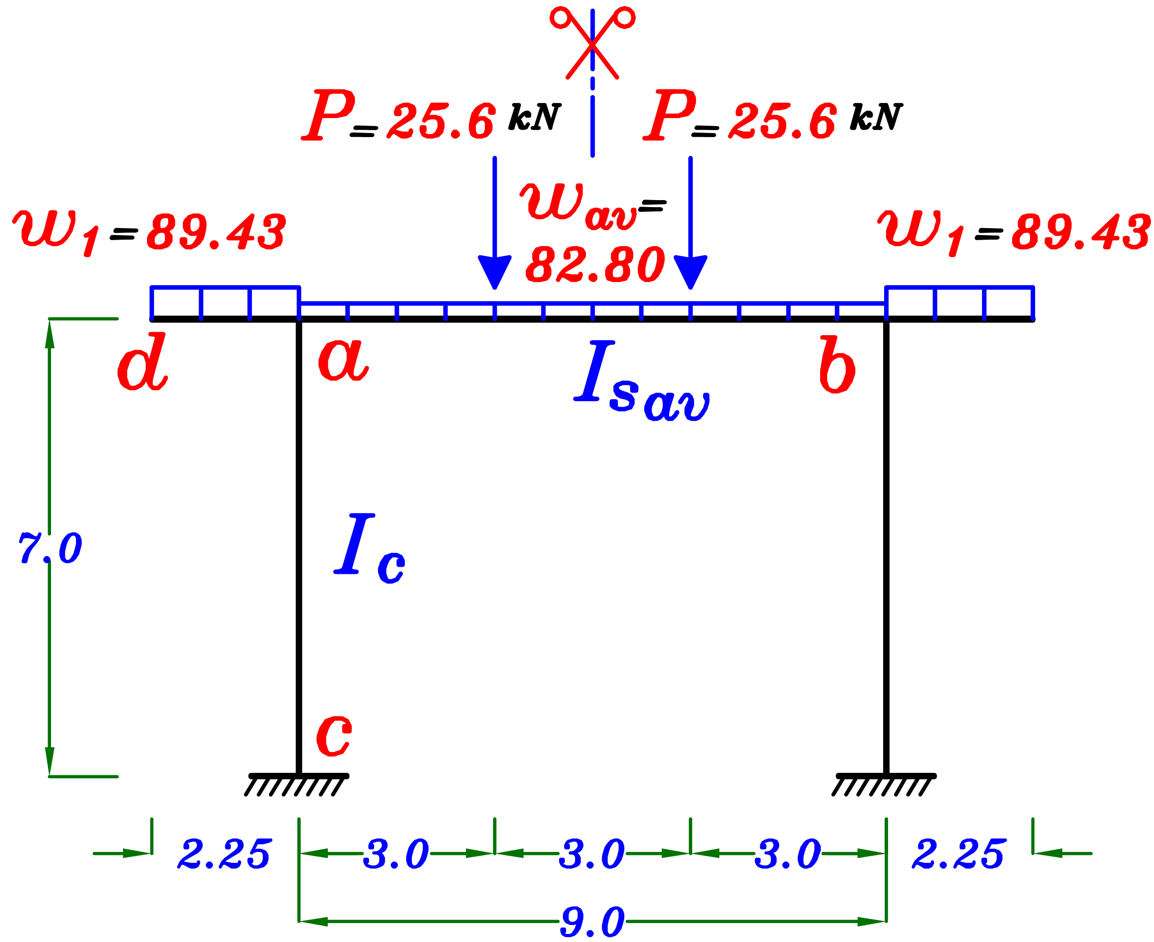
عليه حمل منتظم واحد





$$w_{av} = \frac{89.43 * 6.0 + 69.56 * 3.0}{9.0} = 82.80$$

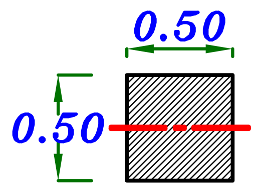




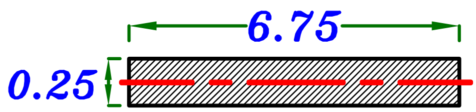
@ Calculate Moment of Inertia For Slabs & Columns.

عمود خارجى

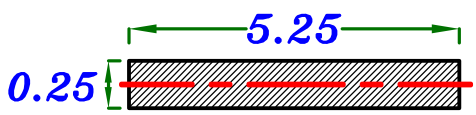
$$I_c = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.50 * 0.50^3}{12} = 3.125 * 10^{-3}\ m^4$$



$$I_{S1} = \frac{L_2 * t_s^3}{12} = \frac{6.75 * 0.25^3}{12} = 8.79 * 10^{-3}\ m^4$$



$$I_{S2} = \frac{L_2 * t_s^3}{12} = \frac{5.25 * 0.25^3}{12} = 6.83 * 10^{-3}\ m^4$$



$$I_{Sav} = \frac{8.79 * 10^{-3}(6.0) + 6.83 * 10^{-3}(3.0)}{9.0} = 8.13 * 10^{-3}\ m^4$$

ⓑ Calculate the stiffness For each member.

$$K_{ab} = \frac{1}{2} * \frac{I_{sav}}{L} = \frac{1}{2} * \frac{8.13 * 10^{-3}}{9.0} = 4.516 * 10^{-4}$$

$$K_{ac} = \frac{I_c}{h} = \frac{3.125 * 10^{-3}}{7.0} = 4.46 * 10^{-4}$$

ⓒ Calculate the Distribution Factors. (D.F.)

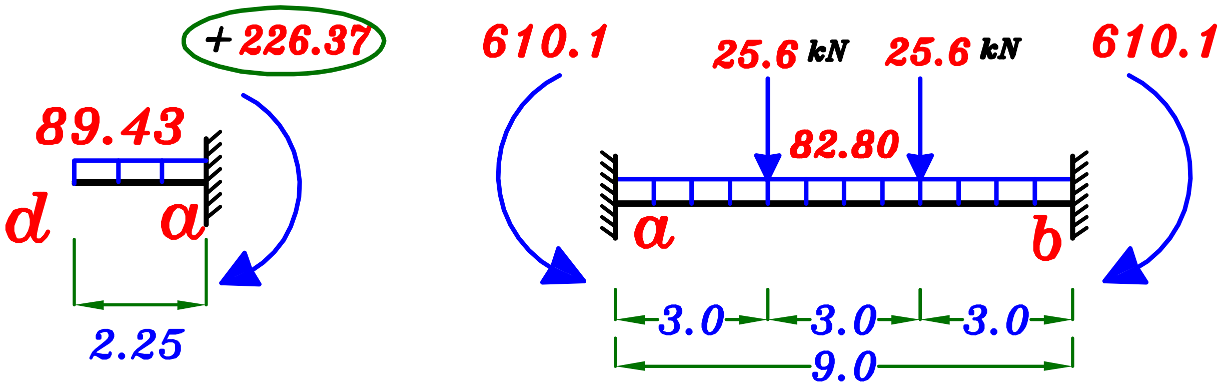
For Joint a

$$\Sigma K = K_{ab} + K_{ac} = 4.516 * 10^{-4} + 4.46 * 10^{-4} = 8.976 * 10^{-4}$$

$$D.F.(ab) = \frac{4.516 * 10^{-4}}{8.976 * 10^{-4}} = 0.503$$

$$D.F.(ac) = \frac{4.46 * 10^{-4}}{8.976 * 10^{-4}} = 0.497$$

ⓓ Calculate Fixed End Moment For the Slab.



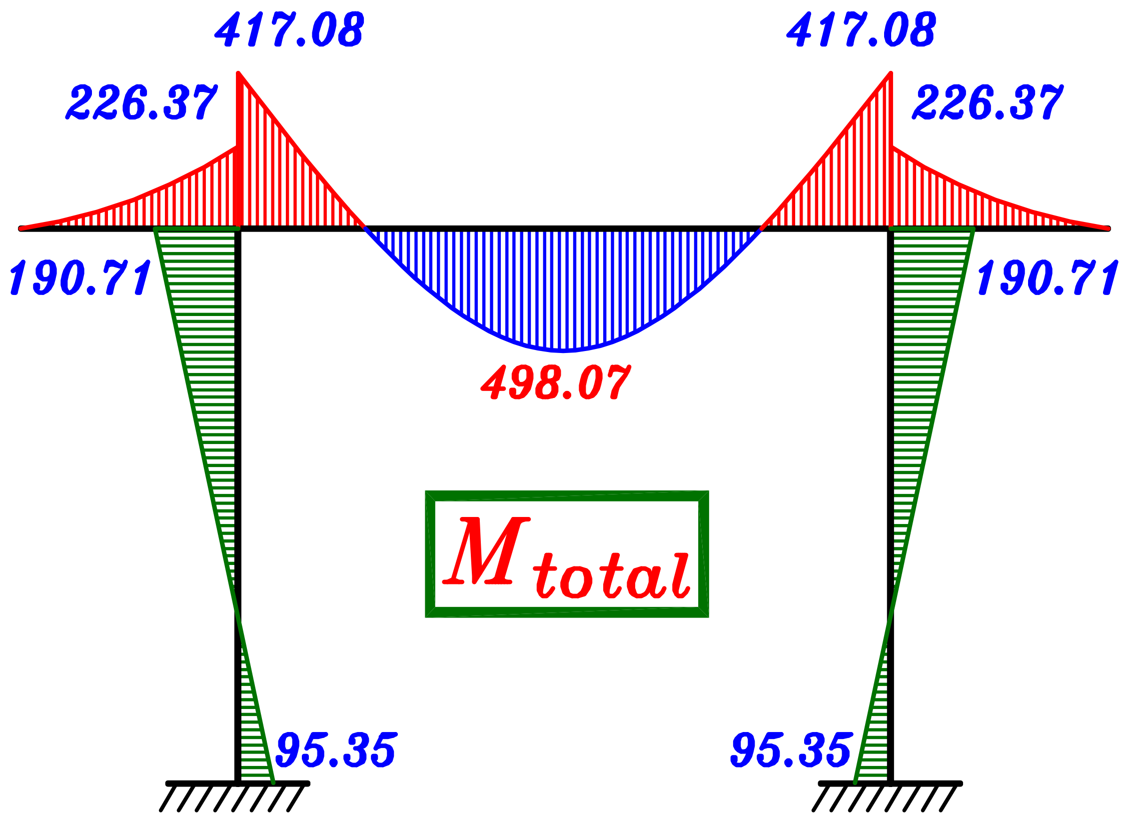
$$F.E.M.(ab) = -\frac{wL^2}{12} - \frac{2}{9} PL = -\frac{82.80 * 9.0^2}{12} - \frac{2}{9}(25.6)(9.0)$$

$$= -610.1 \text{ kN.m.}$$

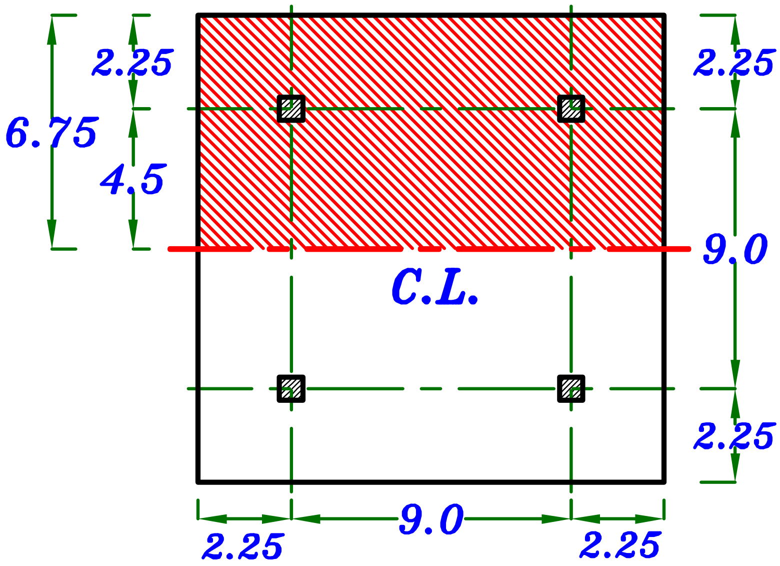
$$F.E.M.(ba) = +\frac{wL^2}{12} + \frac{2}{9} PL = +610.1 \text{ kN.m.}$$

$$F.E.M.(ad) = +\frac{wL^2}{2} = +\frac{89.43 * 2.25^2}{2} = +226.37 \text{ kN.m.}$$

Joint	c	a		
member	c – a	a–c	a–d	a–b
D.F.	0	0.497	0	0.503
F.E.M.	0	0	+226.37	–610.1
B.M.	0	+190.71	0	+193.02
C.O.M.	+95.35	0	0	0
B.M.	0	0	0	0
M_F	+95.35	+190.71	+226.37	–420.08



Modification Factor.



عرض شريحه التصميم الكليه

Total Strip width =

$$= \frac{9.0}{2.0} + 2.25 = 6.75 \ m$$

$$\boxed{b_{c.s.} = \frac{L_2}{4} + \text{Width of the Cantilever}}$$

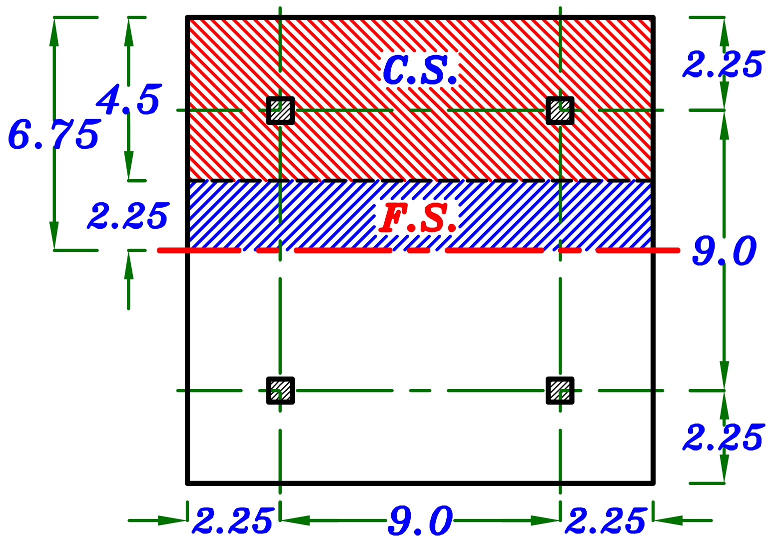
يؤخذ عرض ال Column strip

$$b_{c.s.} = \frac{9.0}{4} + 2.25 = 4.50 \ m$$

$$\boxed{b_{F.S.} = \text{Total Strip width} - b_{c.s.}}$$

و يؤخذ عرض ال Field strip

$$b_{F.S.} = 6.75 - 4.5 = 2.25 \ m$$



Modification Factor For Field Strip

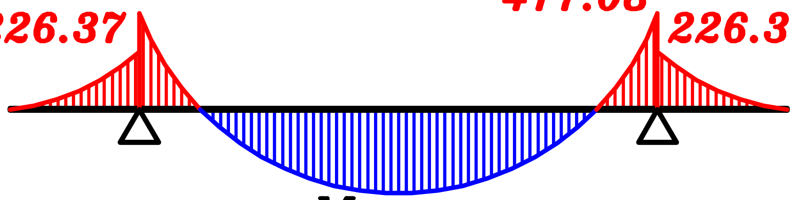
$$\boxed{M.F. = \frac{\text{العرض الحقيقى لل Field strip}}{\frac{1}{2}\ \text{عرض الشريحه الكلى}} = \frac{2.25}{3.375} = 0.667}$$

Distribute the moment of the Frame on Column Strip and Field Strip.

$\boxed{M_{total}}$

$M_2 = 417.08$

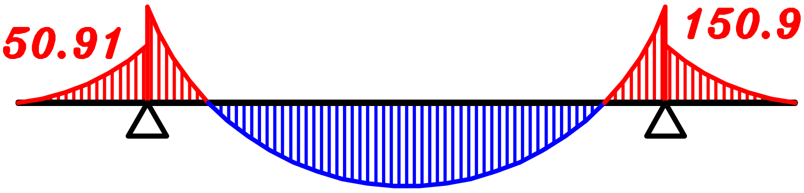
$M_1 = 226.37$ ... 417.08 ... 226.37



$M_3 = 498.07$

$M_1 * \frac{4.5}{6.75} = 150.91$ (عرض ال C.S. / عرض الشريحه كله)

$0.80\ M_2 = 333.66$ ... 333.66 ... 150.91

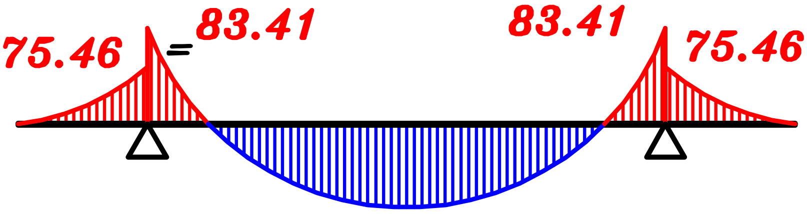


moment C.S.

$0.55\ M_3 = 273.94$

$M_1 * \frac{2.25}{6.75} = 75.46$ (عرض ال F.S. / عرض الشريحه كله)

$0.20\ M_2 = 83.41$ ... 83.41 ... 75.46



moment F.S.

$0.45\ M_3 = 224.13$

$$\text{Modification Factor} = \frac{2.25}{3.375}$$

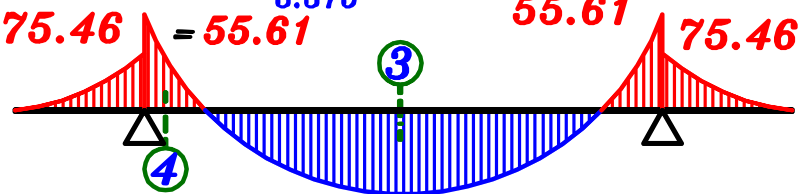
$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{2.25}{3.375}}$$

Modified moment F.S.

$b_{F.S.} = 2.25\ m$

$83.41 * \frac{2.25}{3.375} = 55.61$

75.46 ... 55.61 ... 75.46



$224.13 * \frac{2.25}{3.375} = 149.42$

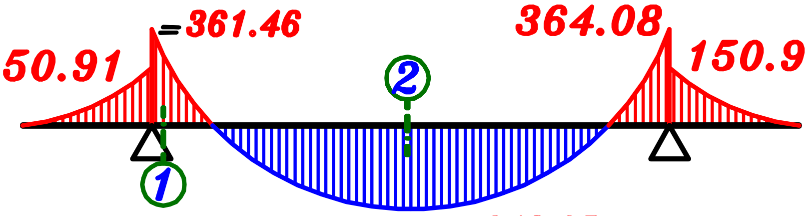
$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

Modified moment C.S.

$b_{C.S.} = 4.50\ m$

$333.66 + 83.41 - 55.61 = 361.46$

150.91 ... 364.08 ... 150.91

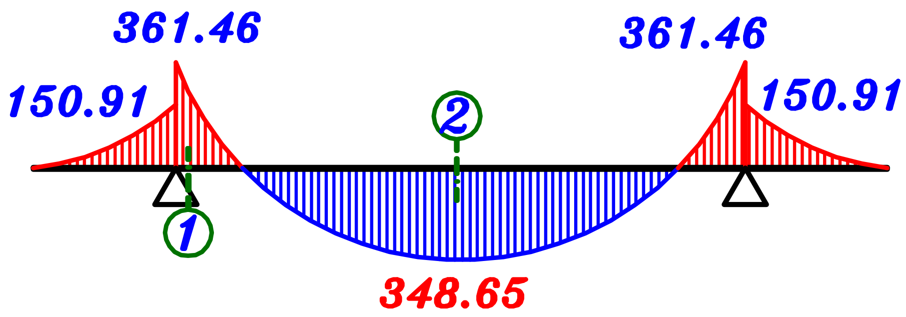


$273.94 + 224.13 - 149.42 = 348.65$

Design the sections of the slab.

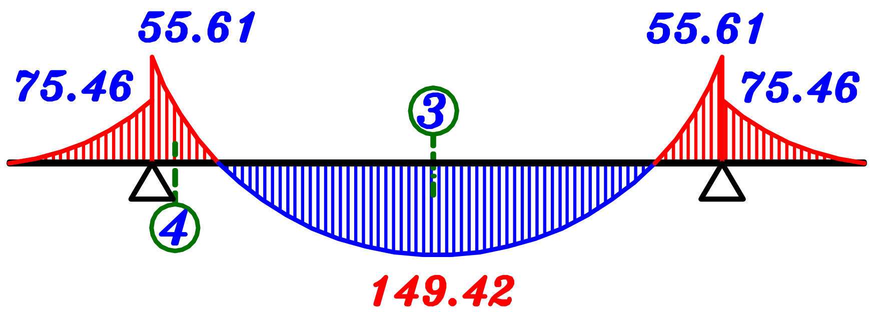
Modified moment C.S.

$b_{C.S.} = 4.50\ m$



Modified moment F.S.

$b_{F.S.} = 2.25\ m$

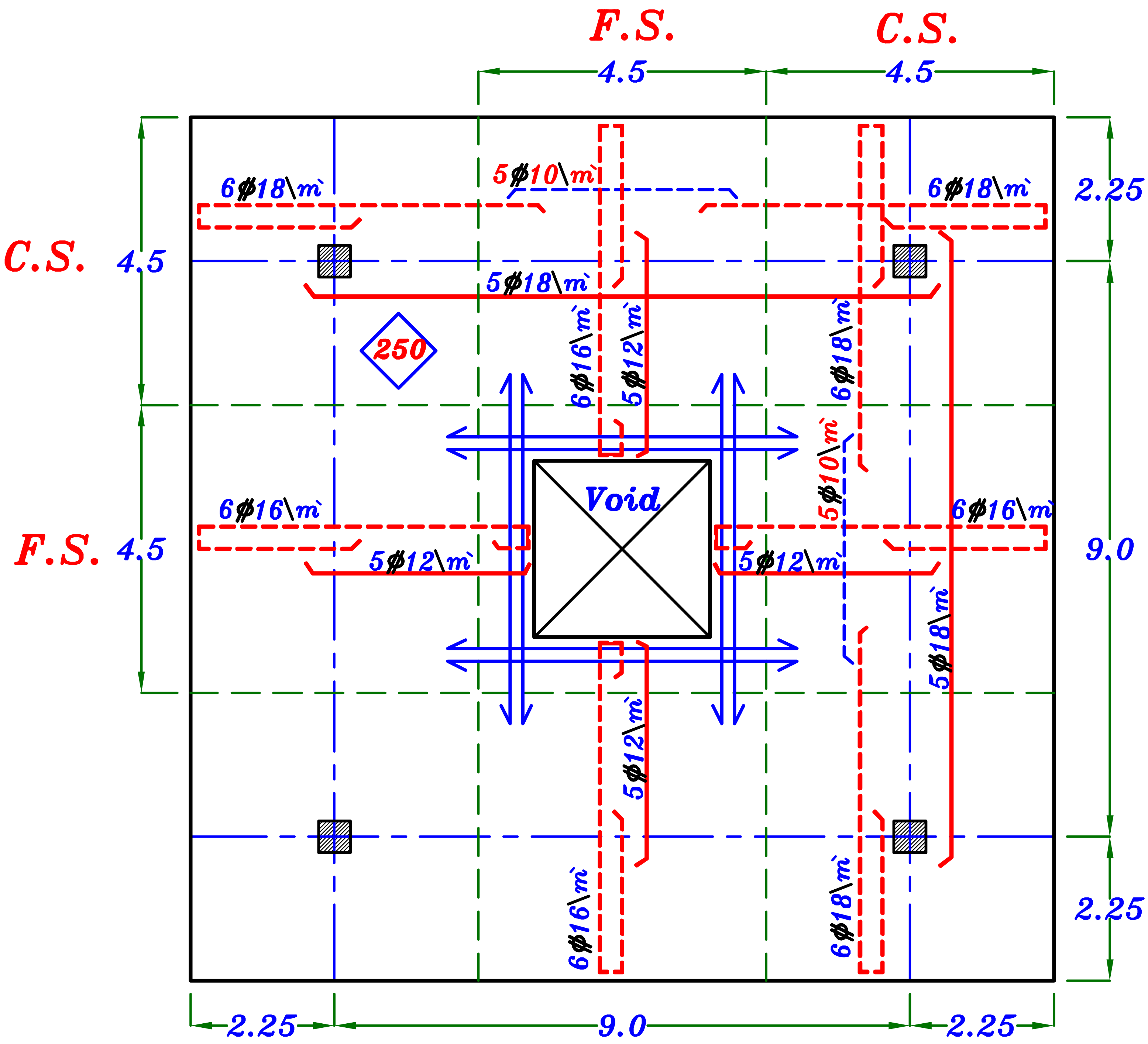


Design of sections.

$d = t_s - 40\ mm = 250 - 40 = 210\ mm$

Strip	Sec.	M (kN.m/strip)	b (mm)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	364.08	4500	210	4.05	0.806	5993	1318	6ϕ18\m
	2	346.55	4500	210	4.13	0.807	5714	1270	5ϕ18\m
Field Strip	3	148.52	2250	210	4.46	0.817	2419	1075	6ϕ16\m
	4	56.04	2250	210	7.31	0.826	890	396	5ϕ12\m

Details of RFT.



Example.

The Figure shows a structural plan of car shed with overall dimensions of $24.0*12.0$ m and no interior beams. The shed is supported on 5 rectangular columns at the center of the shed with dimensions of (350*800 mm) and a height of 5 m and is supported at the edges using the shown reinforced concrete tie.

Data.

$F_{cu} = 35\ N\backslash mm^2$ $\quad F_y = 400\ N\backslash mm^2$

Req.

$F.C. = 3.0\ kN\backslash m^2$, $L.L. = 1.0\ kN\backslash m^2$

① Check punching shear of the slab at column C_1.

② Calculate the internal Forces (B.M. & N.F.) For an intermediate Frame using the structural system and the given Force.

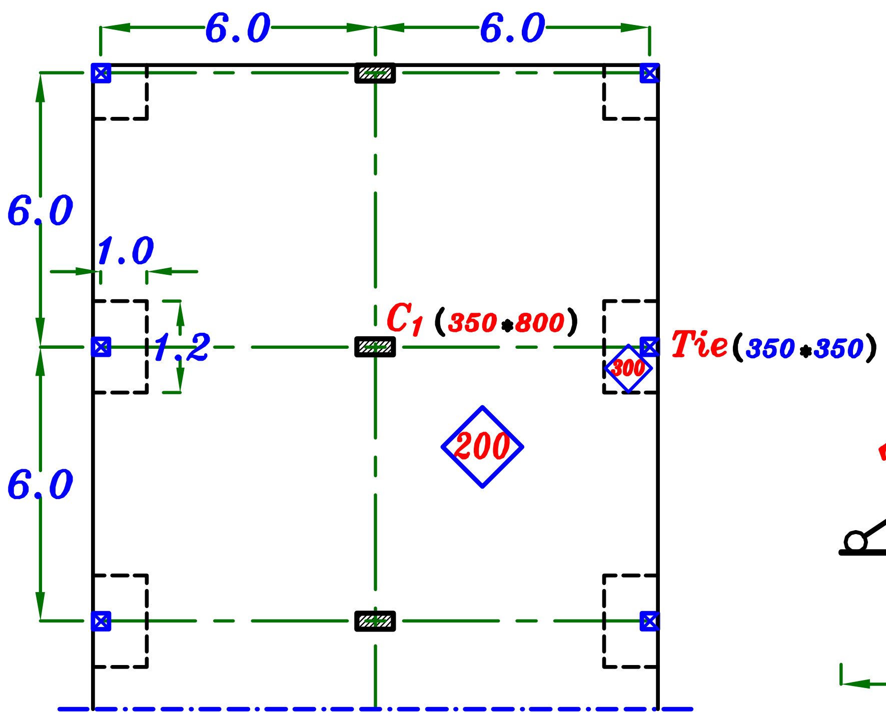
③ Design the Flat slab in X–Direction.

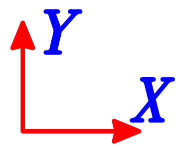
④ Design the RC. tie and draw its details of RFT. at the marked joint A.

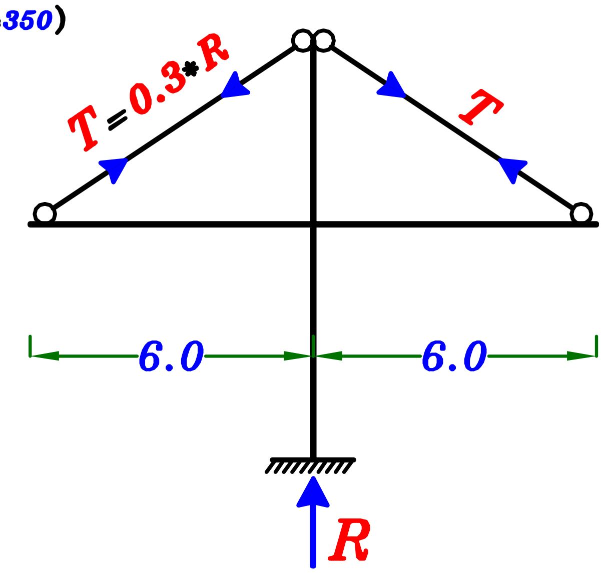
⑤ Design the column C_1

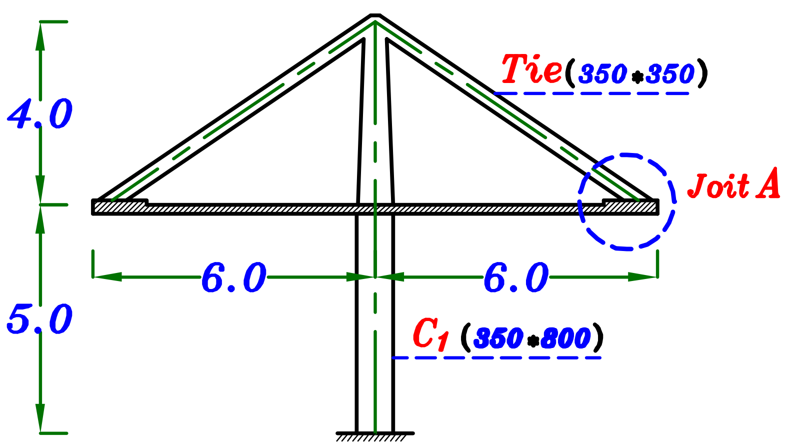
⑥ Draw a half plan showing details of reinforcement of the slab.

(The reinforcement in Y–Direction should be reasonably assumed without calculations.)









Solution.

1-Concrete Dimensions.

Column dimensions. (350*800) as given in data.

Slab Thickness $\boxed{t_s = 200\,mm}$ as given in data.

Drop panel Thickness $\boxed{t_d = 300 - 200 = 100\,mm}$

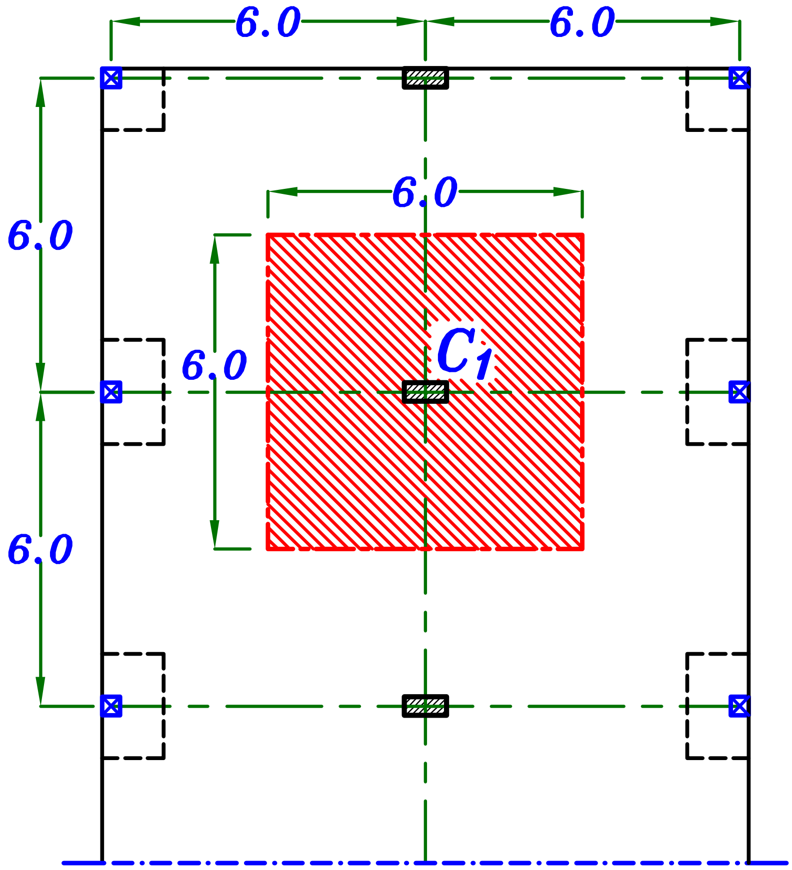
2-Loads on the Slab.

$$w_s = 1.4\left[(t_s)\gamma_c + F.C.\right] + 1.6(L.L.)$$

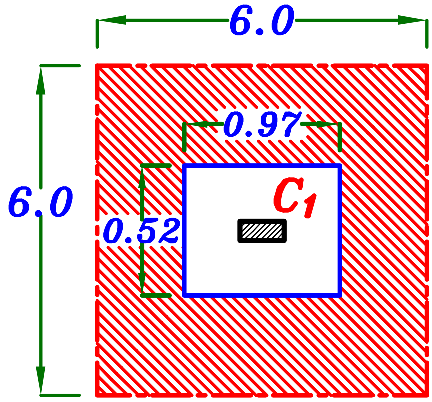
$$w_s = 1.4\left[(0.20)*25 + 3.0\right] + 1.6(1.0) = 12.80\ kN\backslash m^2$$

3-Check Punching on interior column

كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى



$\underline{\underline{C_1}}$ Interior Column.



$$d = t_s - 30\,mm = 200 - 30 = 170\ mm = 0.17\,m$$

$$C_1 + d = 0.80 + 0.17 = 0.97\ m$$

$$C_2 + d = 0.35 + 0.17 = 0.52\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 12.80\left[6.0 * 6.0 - 0.97 * 0.52\right] = 454.34\ kN$$

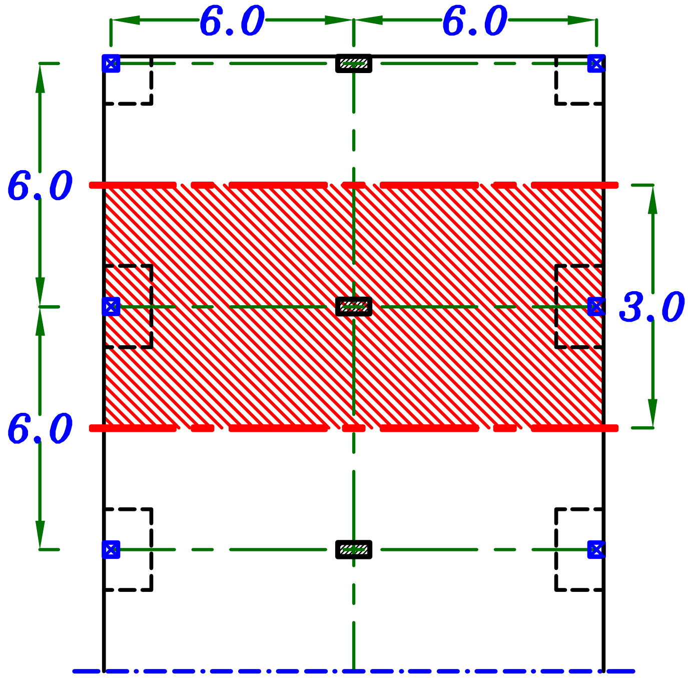
$$A_p = (b_o * d) = (2*970 + 2*520) * 170 = 506600\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{454.34 * 10^3}{506600} * 1.15 = 1.03\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{35}{1.5}} = 1.52\ N/mm^2$$

$$\boxed{q_{pu} < q_{pcu}} \longrightarrow \text{Safe punching}$$

X-Direction.



$$Span = L_1 = 6.0\ m$$

$$Width = L_2 = 6.0\ m$$

$$b_{C.S.} = \frac{L_2}{2} = 3.0\ m$$

$$b_{F.S.} = L_1 - \frac{L_2}{2} = 3.0\ m$$

$$w = w_s * L_2 = 12.80 * 6.0$$

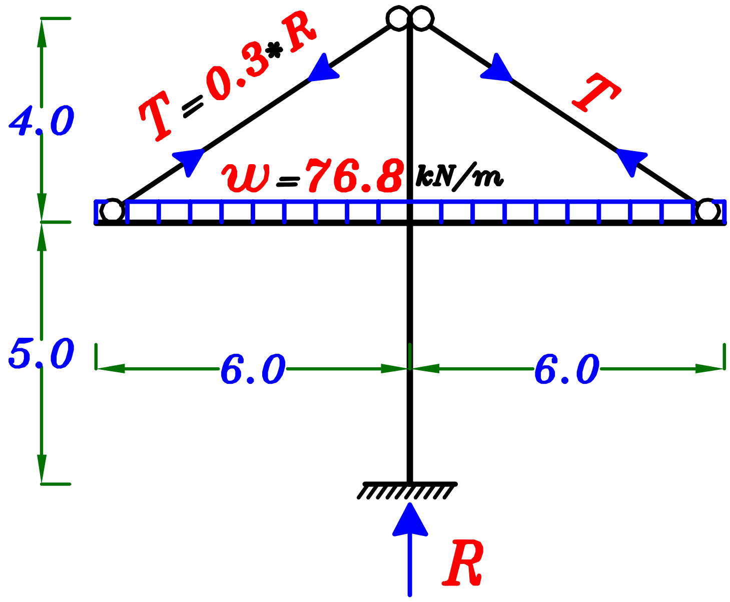
$$= 76.8\ kN/m$$

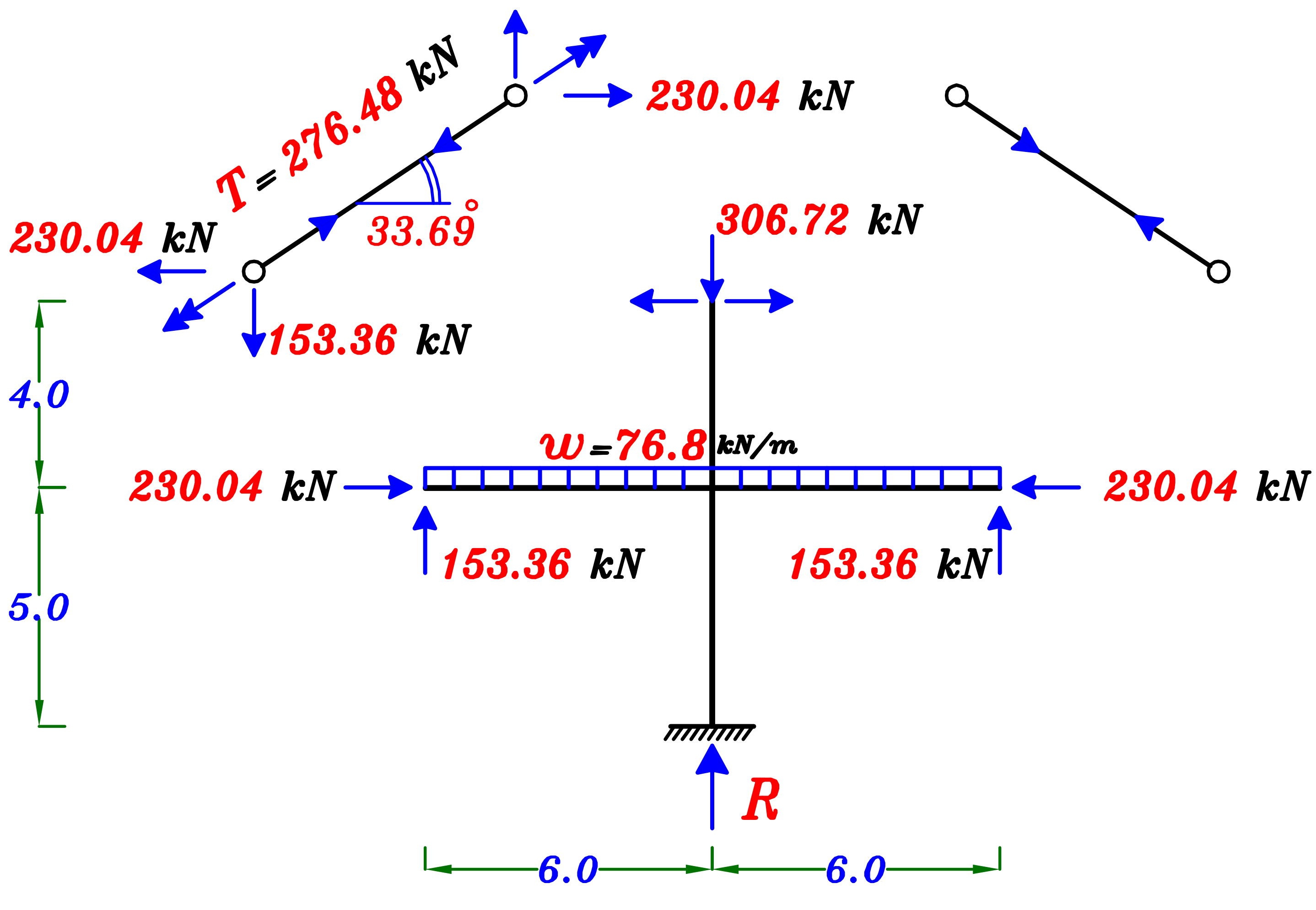
هذه الحاله لا تحتاج للحل بال **moment distribution**
لان العلاقه بين **R & T** معطاه أى أنه **determinate Frame**

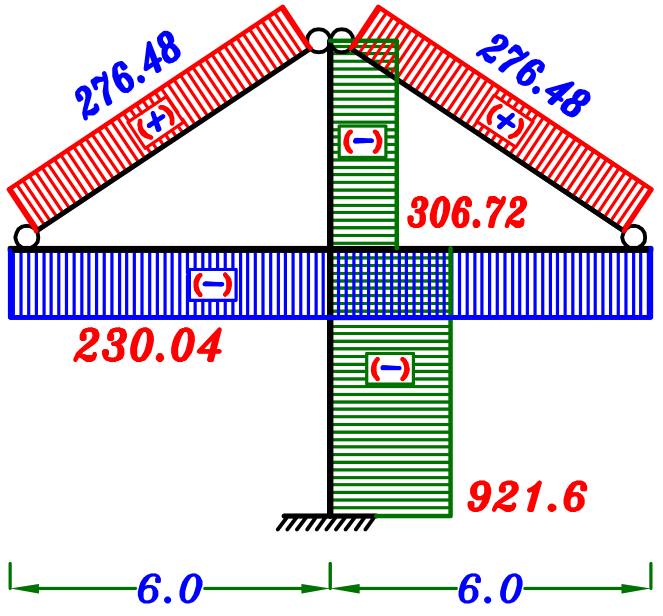
ملحوظه اذا لم تكن العلاقه معطاه سيكون **indeterminate Frame**
و سنضطر لحله بطريقه **Virtual work** نظرا لوجود **Link member**

$$R = 76.8 * 12.0 = 921.6 \text{ kN}$$

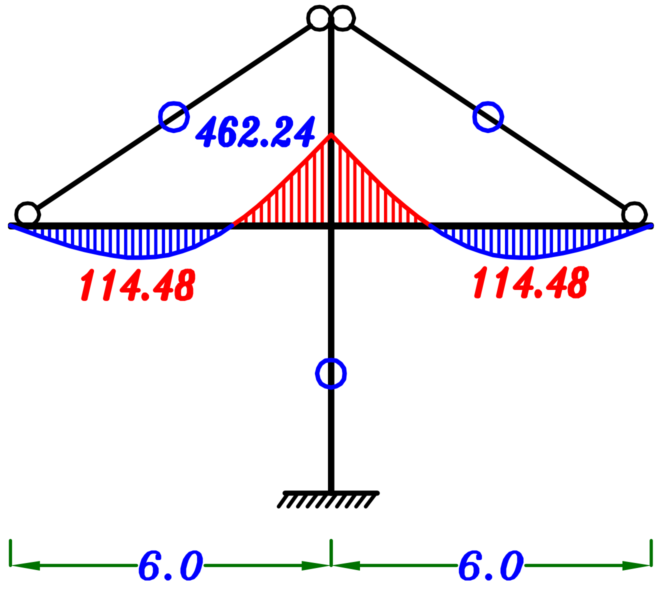
$$T = 0.3 * R = 0.3 * 921.6$$
$$= 276.48 \text{ kN}$$







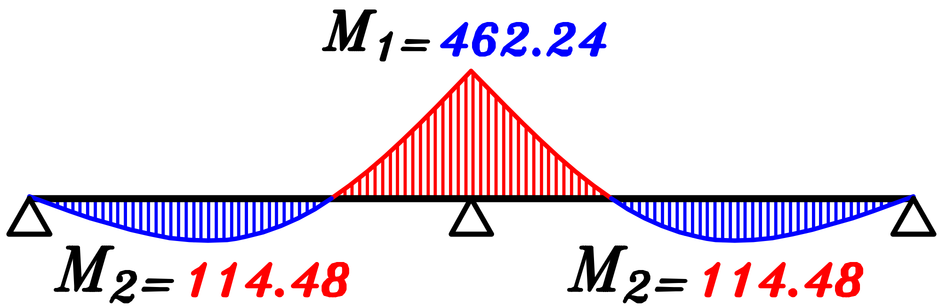
N.F.D.



M_{total}

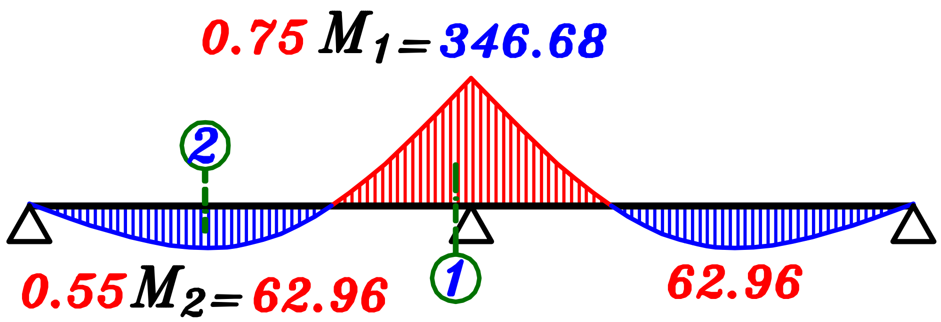
$$b_{C.S.} = b_{F.S.} = \frac{L_2}{2} = 3.0\ m \longrightarrow \text{No Modification Factor}$$

M_{total}



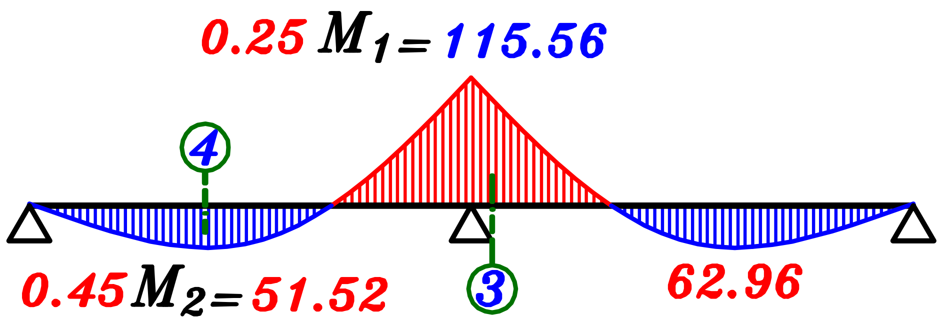
moment C.S.

$b_{C.S.} = 3.0\ m$



moment F.S.

$b_{F.S.} = 3.0\ m$



Design of sections.

$$\frac{P}{F_{cu}\,b\,t} = \frac{230.04*10^3}{30*6000*200} = 0.0054 < 0.04 \longrightarrow \text{Neglect } P$$

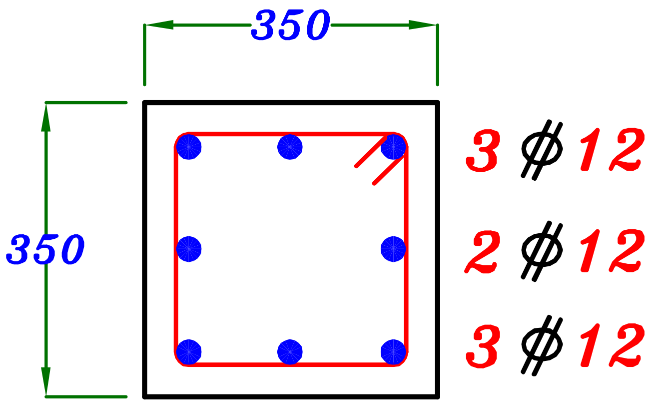
$$d = t_s - 40\,mm = 200 - 40 = 160\,mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm^2/b)	A_s (mm^2/m)	No. of bars/m
Column Strip	1	346.68	3000	160	2.78	0.717	7554	2518	10ϕ18\m
	2	62.96	3000	160	6.50	0.826	1190	396.6	5ϕ12\m
Field Strip	3	115.56	3000	160	4.82	0.825	2188	729.3	7ϕ12\m
	4	51.52	3000	160	7.22	0.826	974	324	5ϕ12\m

④ **Design the RC. tie and draw its details of RFT. at the marked joint A.**

Tie (350 * 350) $\qquad T_{U.L.} = 276.48\ kN$

$$A_s = \frac{T_{U.L.}}{F_y/\gamma_s} = \frac{276.48*10^3}{400/1.15} = 794.88\ mm^2 \qquad \boxed{8\phi12}$$



⑤ Design the column C_1

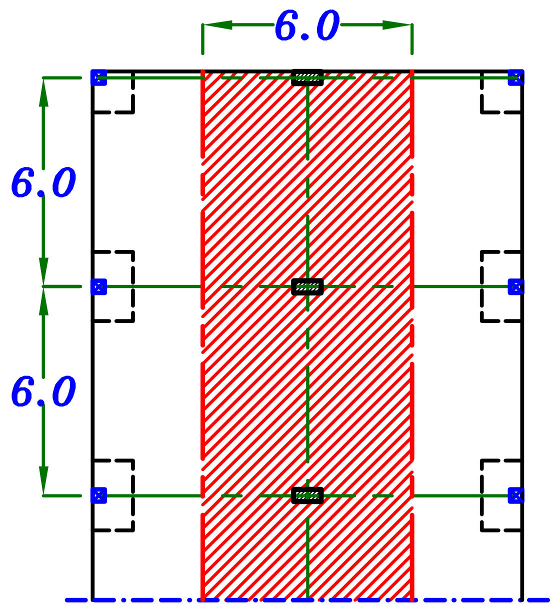
لحساب العزم على العمود يكون نسبه من عزم شريحه البلاطه فى الاتجاه الطويل .

$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8}$$

$$= \frac{(12.80 * 6.0)\left(6.0 - \frac{2}{3} * 0.35\right)^2}{8}$$

$$\boxed{M_o = 319.24 \ kN.m}$$

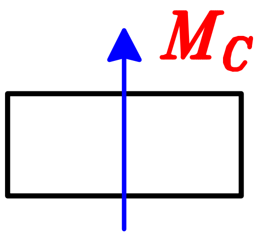
Long Direction



$$M_C = 50\ \%\ M_{c.s.} = 0.5 * (0.50\ M_o)$$

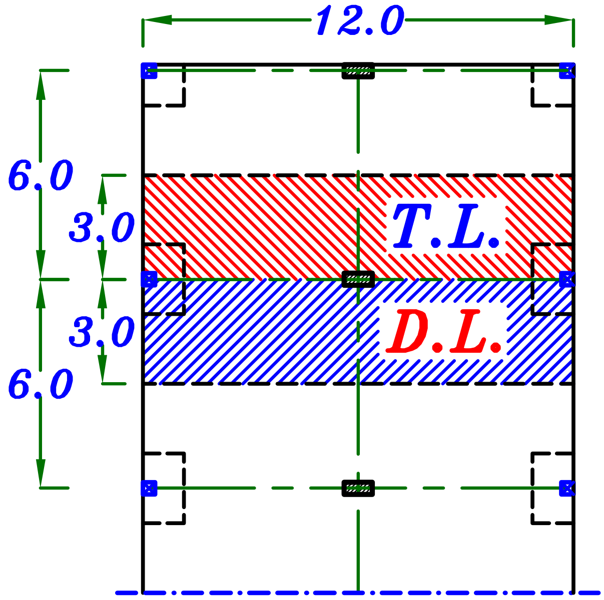
$$= 0.5 * (0.50 * 319.24)$$

$$= 79.81\ kN.m$$



$$g_s = 0.9\left[(t_s)\gamma_c + F.C.\right]$$

$$g_s = 0.9\left[(0.20) * 25 + 3.0\right] = 7.2 \quad kN\backslash m^2$$

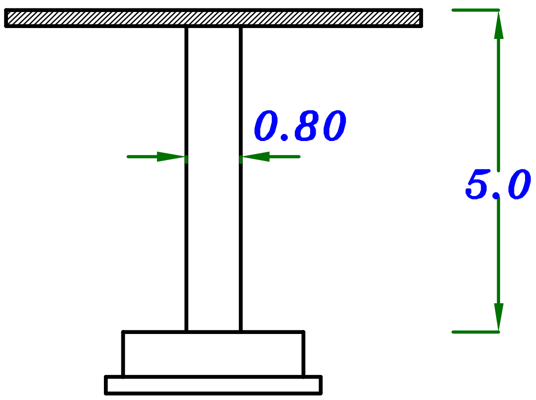


$$P = \left[w_s * \left(\frac{L_1}{2} * L_2\right) + g_s * \left(\frac{L_1}{2} * L_2\right)\right] * 1.1$$

$$P = \left[12.80 * (3.0 * 12.0) + 7.2 * (3.0 * 12.0)\right] * 1.1 = 792.0\ kN$$

Check Buckling.

① In plane.



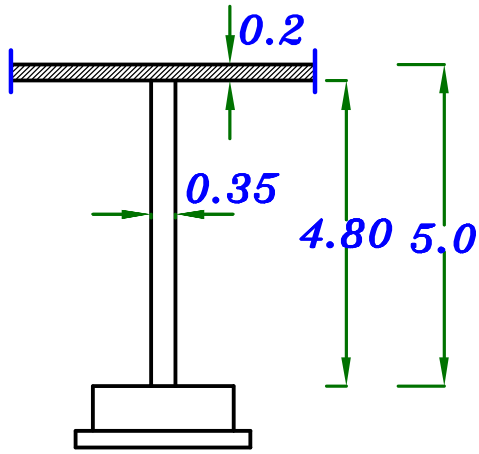
Upper Case ④ } $k = 2.2$
Lower Case ①

$H_o = 5.0$ m

$$\lambda_b = \frac{2.2 * 5.0}{0.8}$$

$$= 13.75 > 10$$

② Out of plane.



Upper Case ② } $k = 1.3$
Lower Case ①

$H_o = 4.8$ m

$$\lambda_b = \frac{1.3 * 4.8}{0.35}$$

$$= 17.8 > 10$$

Take the bigger value of $\lambda_b = 17.8$ (Out of plane.)

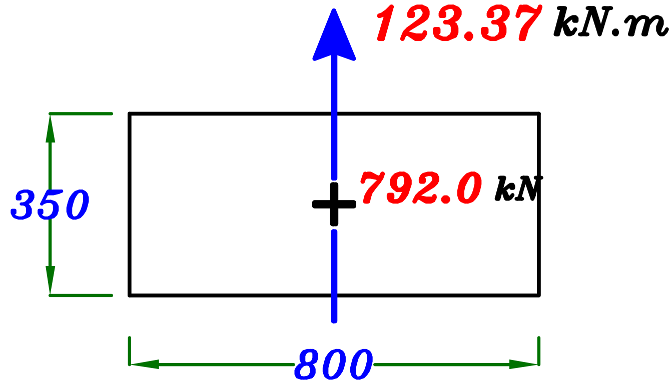
The Buckling is Out of plane.

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{17.8^2 * 0.35}{2000} = 0.055 \text{ m}$$

$$M_{add.} = P * \delta = 792.0 * 0.055 = 43.56 \text{ kN.m}$$

$$M_{Total} = M_{ext.} + M_{add} = 79.81 + 43.56 = 123.37 \text{ kN.m}$$

$$e = \frac{M}{P} = \frac{123.37}{792.0} = 0.155\ m$$



$$\therefore \frac{e}{t} = \frac{0.155}{0.35} \simeq 0.44 \xrightarrow{use} I.D.$$

$$\zeta = \frac{350 - 100}{350} = 0.714 = 0.70 \xrightarrow{use} \text{ECCS Design Aids Page } 4\text{–}22$$

$$\left.\begin{array}{l} \dfrac{P_U}{F_{cu}\, b\, t} = \dfrac{792.0 * 10^3}{35 * 800 * 350} = 0.081 \\[2ex] \dfrac{M_U}{F_{cu}\, b\, t^2} = \dfrac{123.37 * 10^6}{35 * 800 * 350^2} = 0.036 \end{array}\right\} \rho = 2.5$$

$$\mu = \rho * F_{cu} * 10^{-4} = 2.5 * 30 * 10^{-4} = 8.75 * 10^{-3}$$

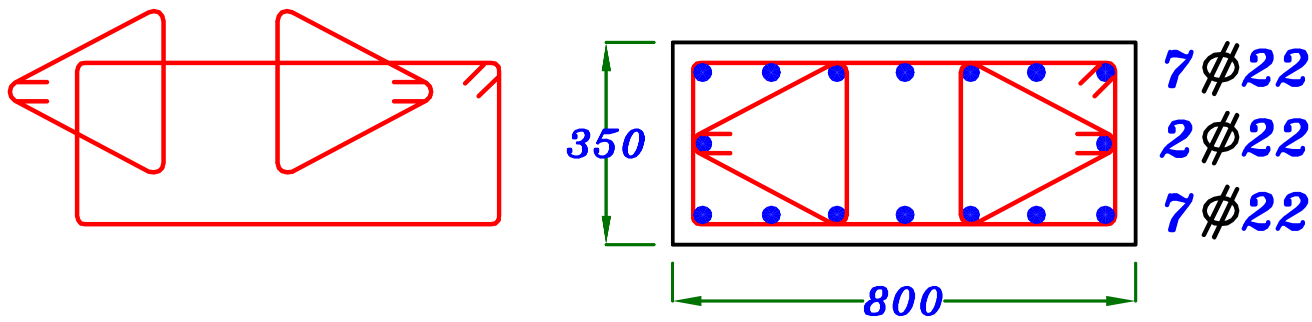
$$A_s = A_s' = \mu * b * t = 8.75 * 10^{-3} * 800 * 350 = 2450\ mm^2$$

$$A_{s\,Total} = A_s + A_s' = 2 * 2450 = 4900\ mm^2$$

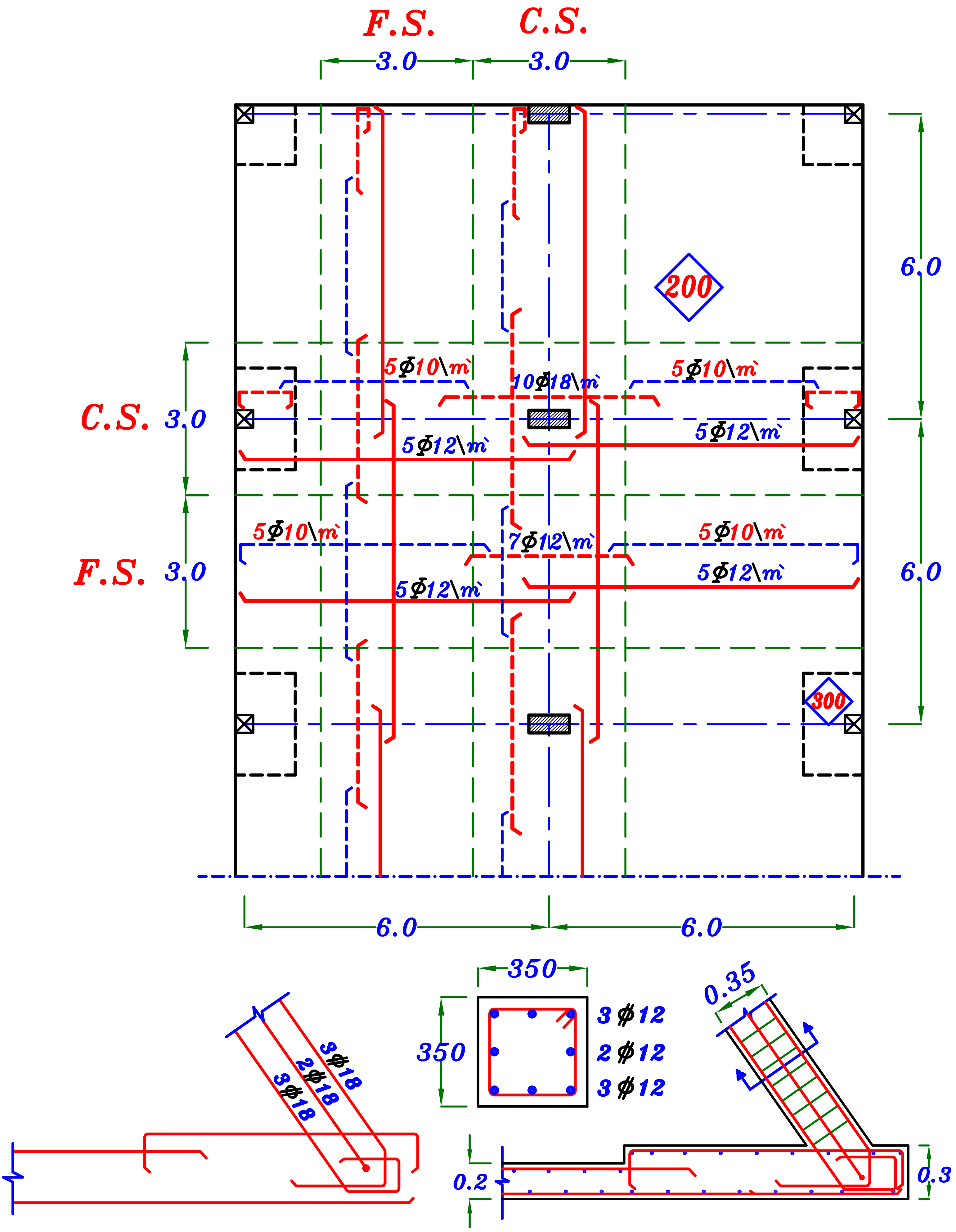
$$A_{s_{min}} = \frac{0.25 + 0.052\,\lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\,(17.8)}{100} * 800 * 350 = 3291.7\ mm^2 < A_{s\,total}$$

$$\text{Take } A_s = A_s' = \frac{A_{s\,Total}}{2} = 2450\ mm^2 \quad \boxed{7\ \phi\ 22}$$



Details of RFT.



Example.

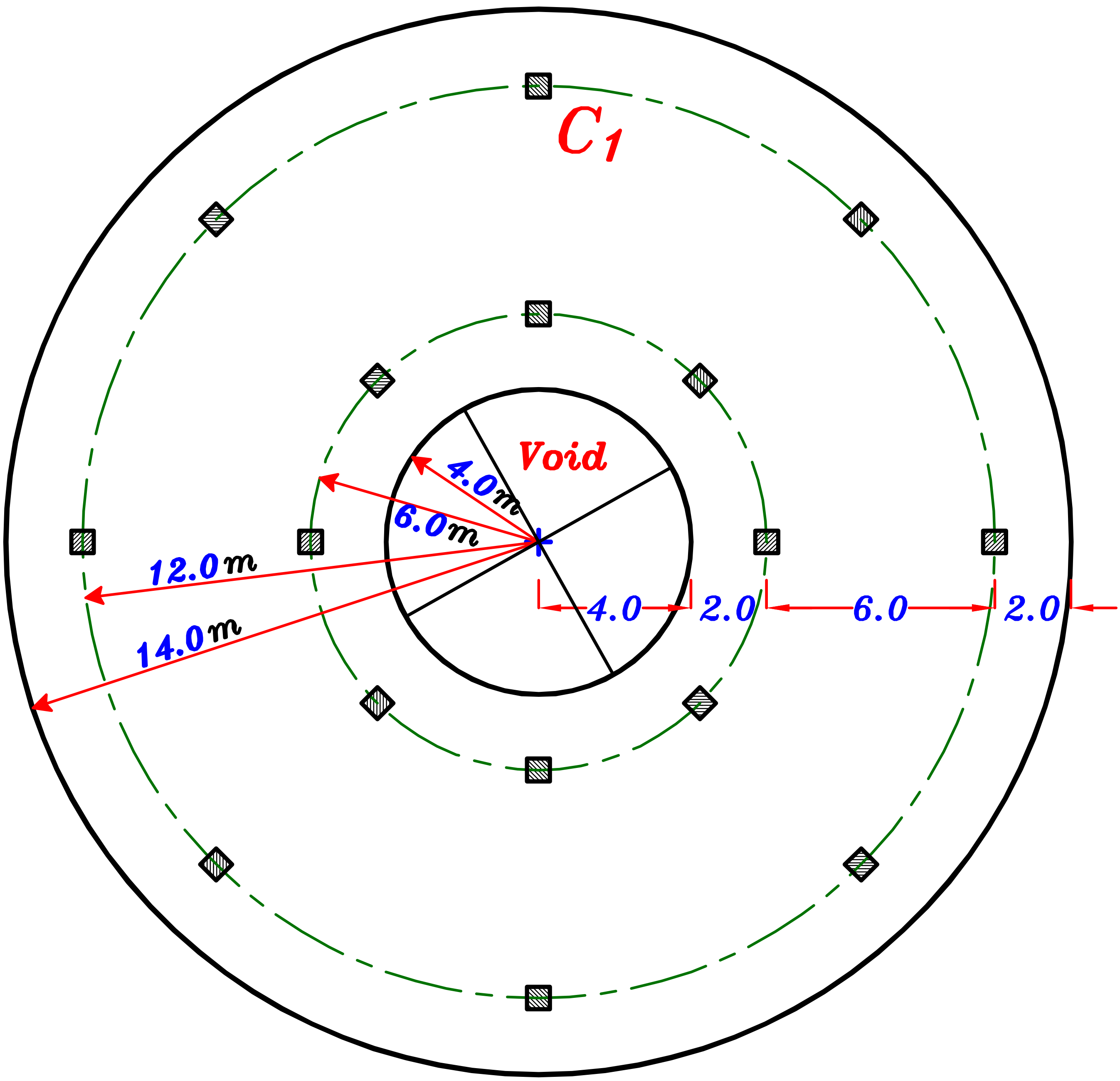
The Figure shows a structural plan of a circular Flat Slab with the given dimensions. The plan For one story with column height = 4.0 m

Data. $F_{cu} = 30\ N\backslash mm^2$, $F_y = 360\ N\backslash mm^2$

$F.C. = 3.0\ kN\backslash m^2$, $L.L. = 1.0\ kN\backslash m^2$

Req.

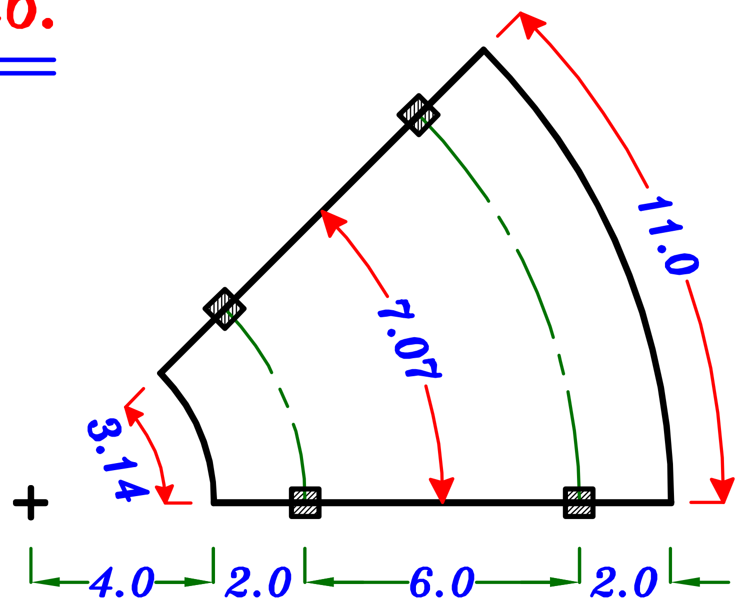
1. Check punching shear of the slab at column C_1.
2. Draw the bending moment in both directions (Meridian and Ring directions)
3. Design the Slab and draw details of RFT. in half Plan.



Sector of the slab.

$$L_1 = L_{av.} = 7.07\,m$$

$$L_2 = 6.0\,m$$



Column dimensions.

$$b_{Col.} = \begin{cases} 300\,mm \\ \dfrac{H}{15} = \dfrac{4000}{15} = 266.6\,mm \\ \dfrac{L_1}{20} = \dfrac{7070}{20} = 353.5\,mm \end{cases}$$

$$\boxed{b_{Col.} = 400\,mm}$$

$(400 * 400)$

Slab Thickness.

Flat Slab.

$$L_1 = 7.07\,m$$

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7070}{36} = 196.4\,mm$

Cantilever $t_s = \dfrac{L_c}{10} = \dfrac{2000}{10} = 200\,mm$

$$\boxed{t_s = 200\,mm}$$

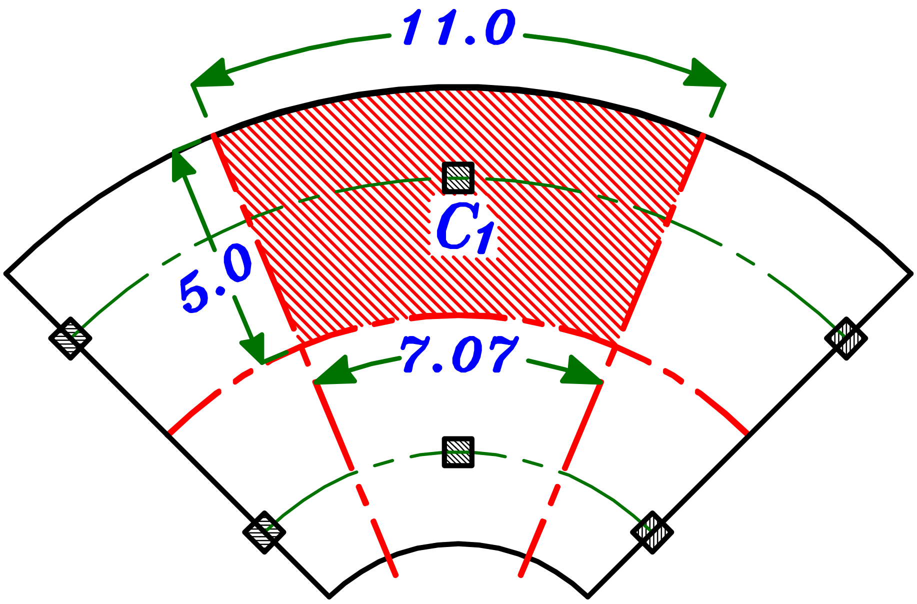
2- Loads on the Slab.

$$w_s = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)$$

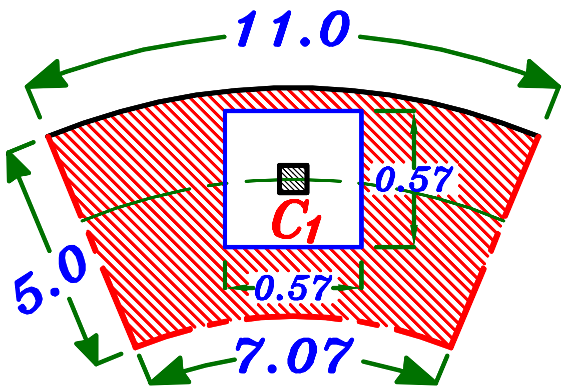
$$w_s = 1.4\,(0.20 * 25 + 3.0) + 1.6\,(1.0) = 12.80\ kN\backslash m^2$$

Check Punching on interior column C_1

كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى



C_1 Interior Column.



$$d = t_s - 30\,mm = 200 - 30$$
$$= 170\ mm = 0.17\ m$$

$$C+d = 0.40 + 0.17 = 0.57\ m$$

$$Q_{pu} = w_s\left[L_1 * L_{av} - (C_1+d)(C_2+d)\right]$$

$$Q_{pu} = 12.80\left[5.0*\left(\frac{7.07+11.0}{2.0}\right) - 0.57*0.57\right] = 574.1\ kN$$

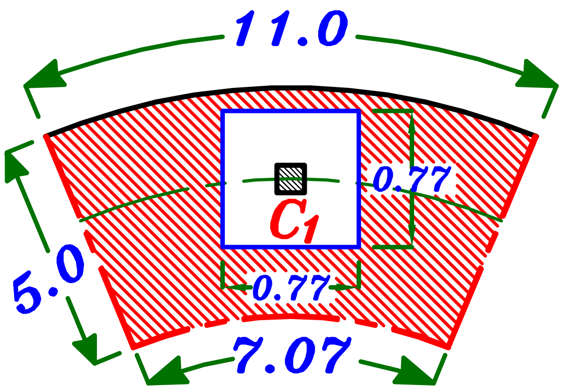
$$A_p = (b_o * d) = (4*570)*170 = 387600\ \ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{574.1 * 10^3}{387600} * 1.15 = 1.70\ \ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{30}{1.5}} = 1.41\ \ N/mm^2$$

$$q_{pu} > q_{pcu} \longrightarrow$$ Unsafe punching
Increase dimensions of the column.

$\underline{C_1}$ Interior Column.



Take the Column $(600 * 600)$

$$d = t_s - 30\,mm = 200 - 30$$
$$= 170\ mm = 0.17\ m$$

$$C+d = 0.60 + 0.17 = 0.77\,m$$

$$Q_{pu} = w_s \left[L_1 * L_{av} - (C_1+d)(C_2+d)\right]$$

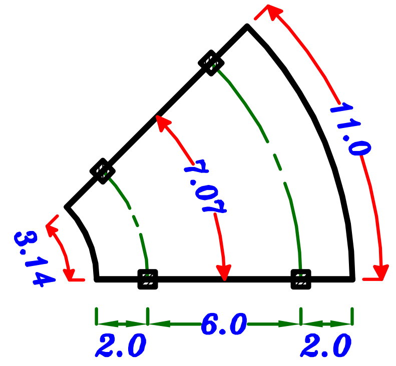
$$Q_{pu} = 12.80 \left[5.0 * \left(\frac{7.07+11.0}{2.0}\right) - 0.77 * 0.77\right] = 570.6\,kN$$

$$A_p = (b_o * d) = (4 * 770) * 170 = 523600\ \ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{570.6 * 10^3}{523600} * 1.15 = 1.25\ N/mm^2$$

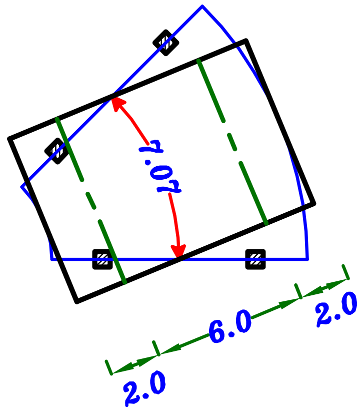
$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{30}{1.5}} = 1.41\ \ N/mm^2$$

$$q_{pu} < q_{pcu} \longrightarrow$$ Safe punching.

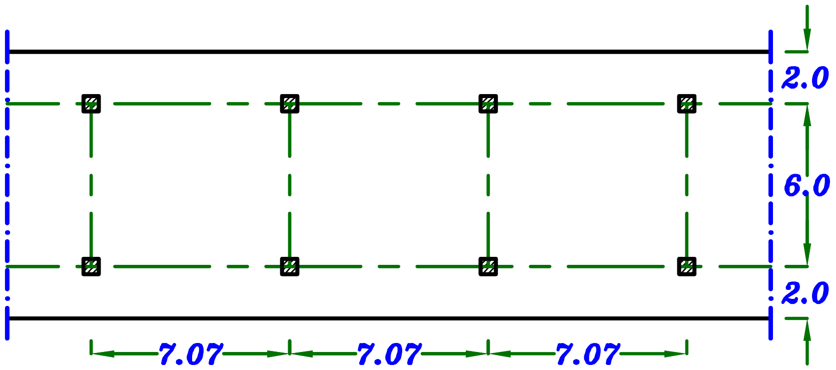


لكى نستطيع حل البلاطه نحسب أبعاد **sector** واحد

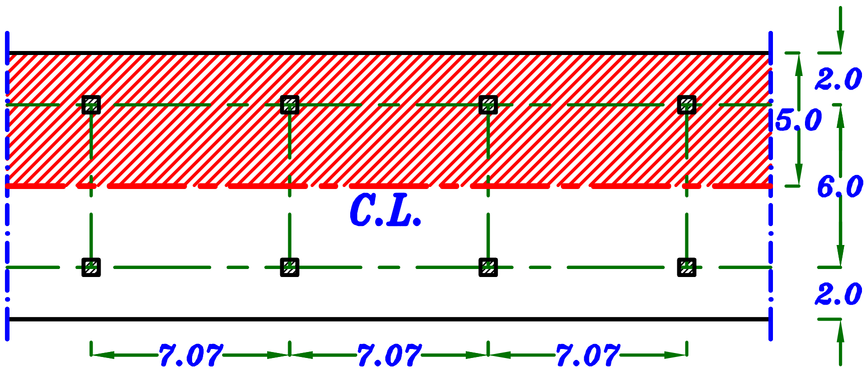
و نحدد الطول المتوسط $L_{av.} = 7.07\,m$



نعتبر ان ال **sector** عباره عن مستطيل

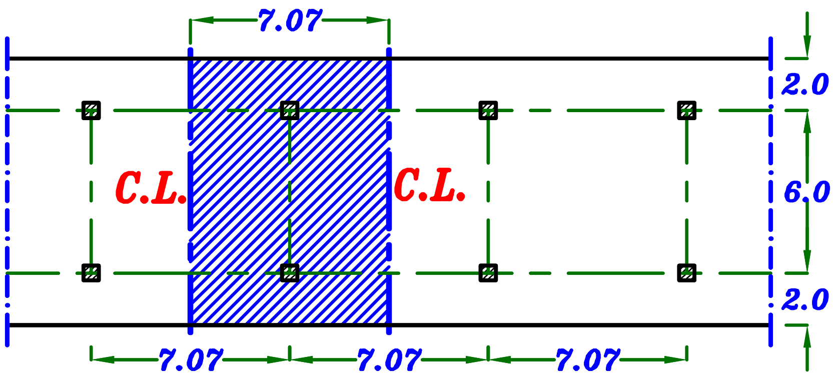


نعتبر ان البلاطه مستقيمه



نأخذ شريحه عرضيه

و نحسب لها ال **moment**

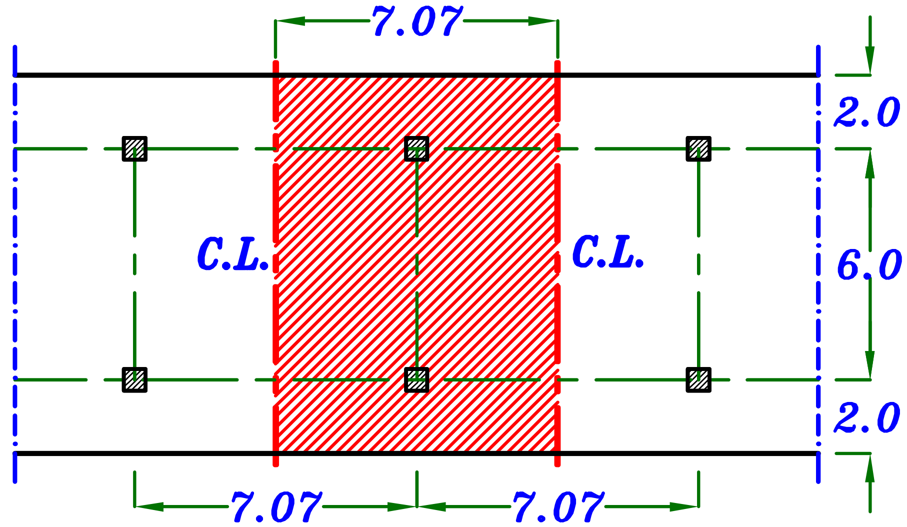


نأخذ شريحه طوليه

و نحسب لها ال **moment**

و بالطبع هذا حل تقريبى

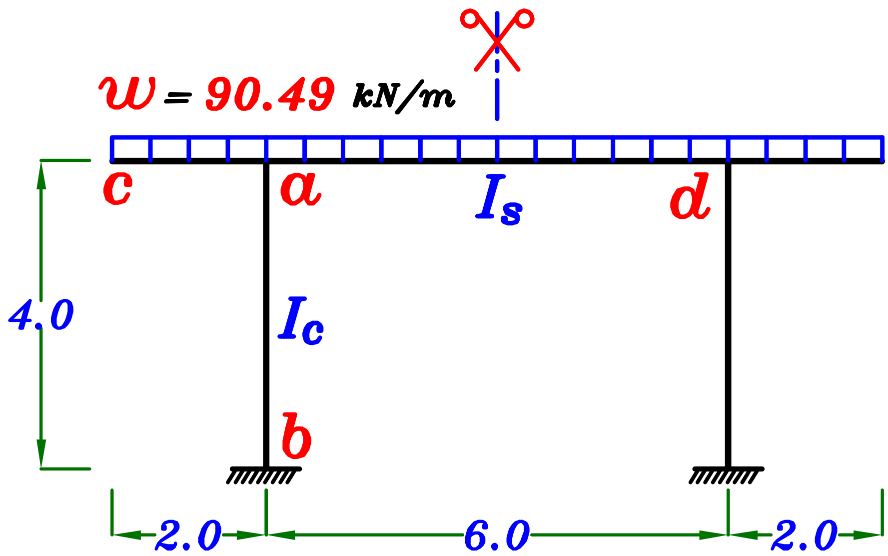
Strip at Short Direction.



$$b_{C.S.} = \frac{L_2}{2} = 3.0 \ m$$

$$b_{F.S.} = L_1 - \frac{L_2}{2} = 7.07 - 3.0 = 4.07 \ m$$

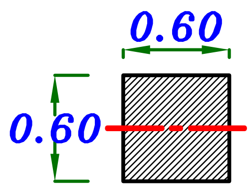
$$w = w_S * L_2 = 12.80 * 7.07 = 90.49 \ kN/m$$



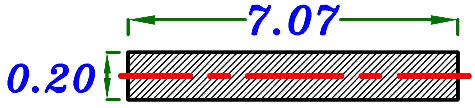
@ Calculate Moment of Inertia For Slabs & Columns.

عمود خارجي

$$I_{c1} = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.60 * 060^3}{12} = 6.48 * 10^{-3} \ m^4$$



$$I_S = \frac{L_{av} * t_s^3}{12} = \frac{7.07 * 0.20^3}{12} = 4.71 * 10^{-3} \ m^4$$



ⓑ Calculate the stiffness For each member.

$$K_{ad} = \frac{1}{2} * \frac{I_s}{L} = \frac{1}{2} * \frac{4.71 * 10^{-3}}{6.0} = 3.925 * 10^{-4}$$

$$K_{ab} = \frac{I_c}{h} = \frac{6.48 * 10^{-3}}{4.0} = 1.62 * 10^{-3}$$

ⓒ Calculate the Distribution Factors. (D.F.)

For Joint a

$$\Sigma K = K_{ad} + K_{ab} = 3.925 * 10^{-4} + 1.62 * 10^{-3} = 2.01 * 10^{-3}$$

$$D.F._{ab} = \frac{1.62 * 10^{-3}}{2.01 * 10^{-3}} = 0.805$$

$$D.F._{ad} = \frac{3.925 * 10^{-4}}{2.01 * 10^{-3}} = 0.195$$

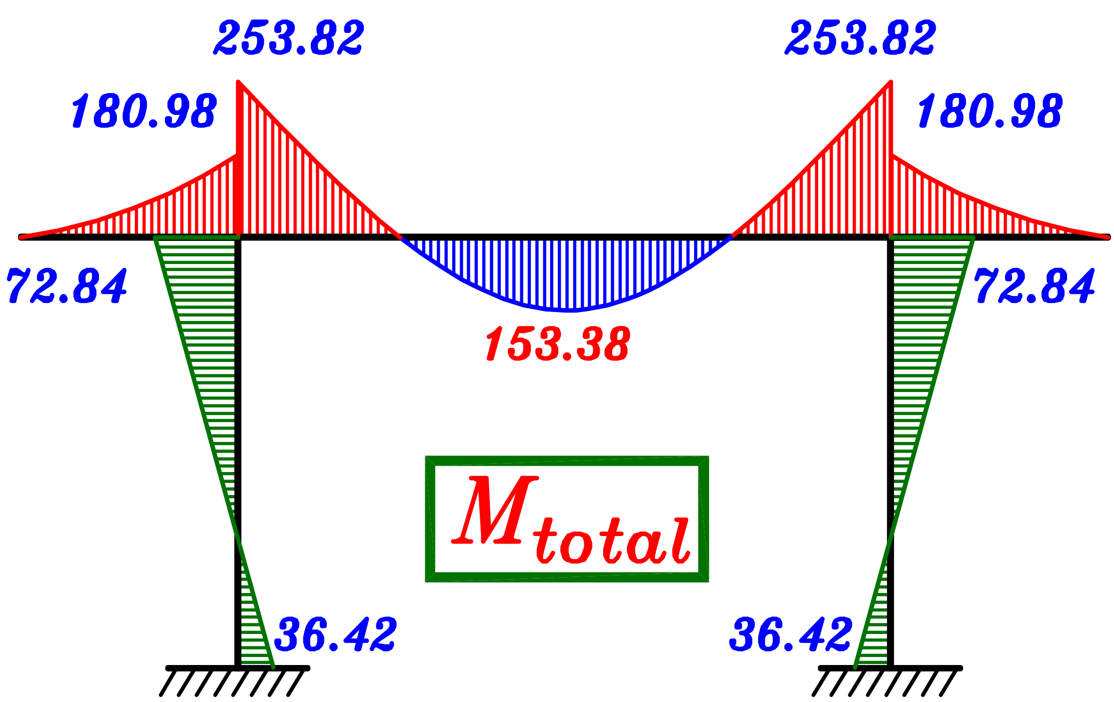
ⓓ Calculate Fixed End Moment For the Slab.

$$F.E.M.(ad) = -\frac{wL^2}{12} = -\frac{90.49 * 6.0^2}{12} = -271.47 \text{ kN.m.}$$

$$F.E.M.(da) = +\frac{wL^2}{12} = +\frac{90.49 * 6.0^2}{12} = +271.47 \text{ kN.m.}$$

$$F.E.M.(ac) = +\frac{wL^2}{2} = +\frac{90.49 * 2.0^2}{2} = +180.98 \text{ kN.m.}$$

Joint	b	a		
member	b-a	a-b	a-c	a-d
D.F.	0	0.805	0	0.195
F.E.M.	0	0	+180.98	-271.47
B.M.	0	+72.84	0	+17.65
C.O.M.	+36.42	0	0	0
B.M.	0	0	0	0
M_F	+36.42	+72.84	+180.98	-253.82



M_{total}

Modification Factor.

Total Strip width = 7.07 m عرض شريحه التصميم الكليه

$$b_{C.S.} = \frac{L_2}{2} = 3.0 \ m$$

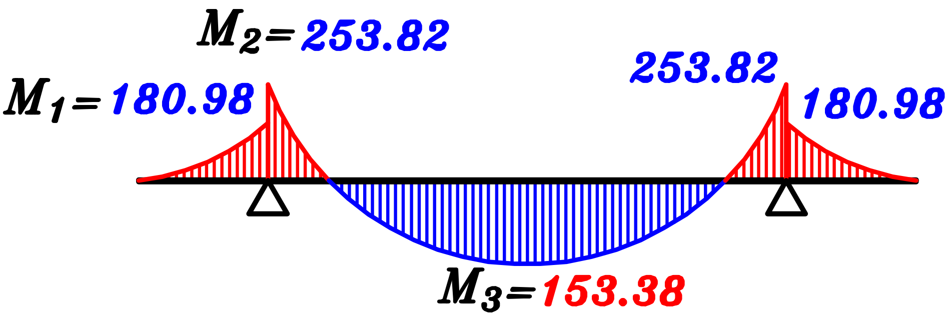
$$b_{F.S.} = L_1 - \frac{L_2}{2} = 7.07 - 3.0 = 4.07 \ m$$

Modification Factor For Field Strip

$$\boxed{M.F. = \frac{\text{Field strip العرض الحقيقى لل}}{\text{عرض الشريحه الكلى } \frac{1}{2}} = \frac{4.07}{3.535} = 1.151}$$

Distribute the moment of the Frame on Column Strip and Field Strip.

M_{total}

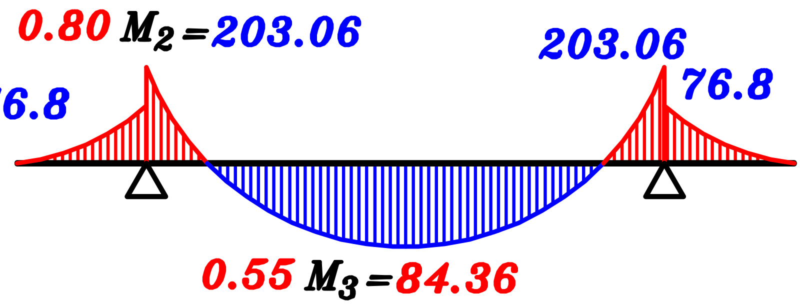


عرض ال C.S.

$M_1 * \frac{3.0}{7.07} = 76.8$

عرض الشريحه كله

moment C.S.

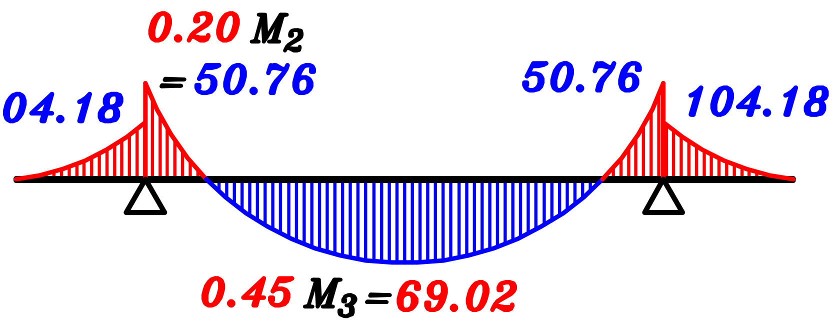


عرض ال F.S.

$M_1 * \frac{4.07}{7.07} = 104.18$

عرض الشريحه كله

moment F.S.

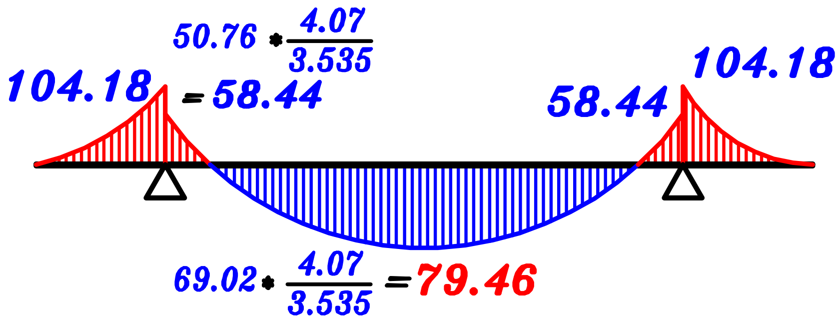


$$\text{Modification Factor} = \frac{4.07}{3.535}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.07}{3.535}}$$

Modified moment F.S.

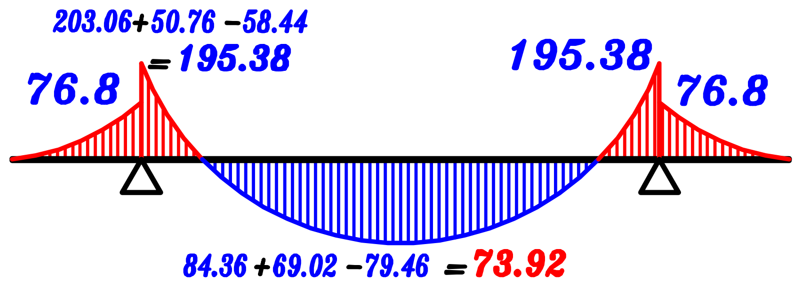
$b_{F.S.} = 4.07\ m$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

Modified moment C.S.

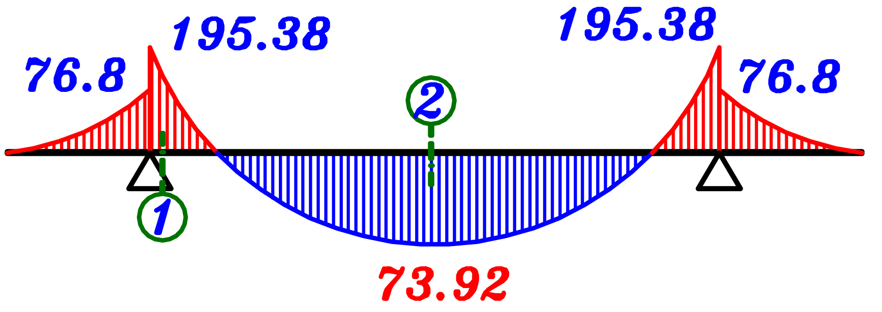
$b_{C.S.} = 3.0\ m$



Design the sections of the slab.

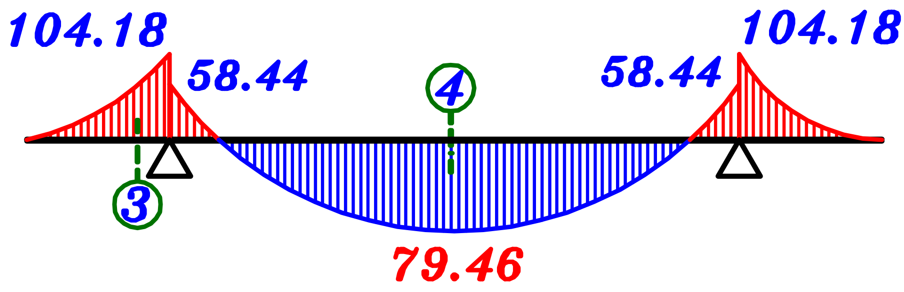
moment C.S.

$b_{C.S.} = 3.0\ m$



moment F.S.

$b_{F.S.} = 4.07\ m$

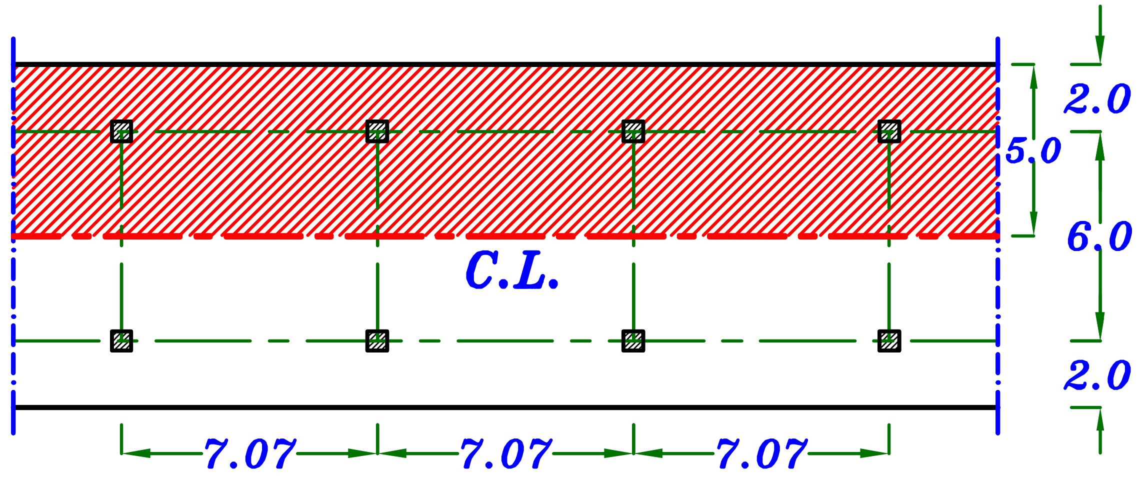


Design of sections.

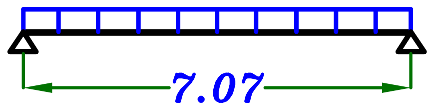
$d = t_s - 40\ mm = 200 - 40 = 160\ mm$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm^2/b)	As (mm^2/m)	No. of bars/m`
Column Strip	1	195.38	3000	160	3.43	0.777	4365	1455	6ϕ18\m
	2	73.92	3000	160	5.58	0.826	1553	517	5ϕ12\m
Field Strip	3	104.18	4070	160	5.47	0.826	2189	537	5ϕ12\m
	4	79.46	4070	160	6.27	0.826	1670	410	5ϕ12\m

Strip at Long Direction.



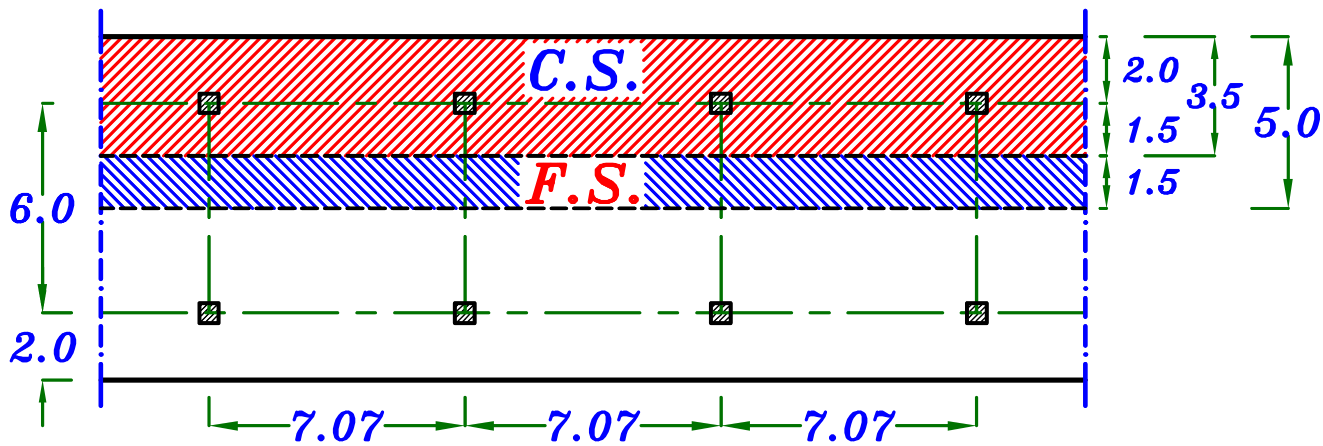
Span $= 7.10\ m$ Width $= 5.0\ m$



$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)^2}{8} = \frac{(12.80 * 5.0)\left(7.07 - \frac{2}{3} * 0.6\right)^2}{8}$$

$$\boxed{M_o = 355.91\ kN.m}$$

Modification Factor.



Total Strip width $= 5.0\ m$

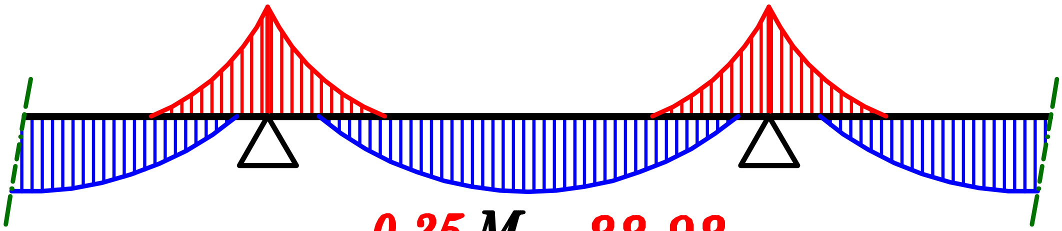
$$b_{C.S.} = \frac{L_2}{4} + \text{Cantilever width} = \frac{6.0}{4.0} + 2.0\ m = 3.5\ m$$

$$b_{F.S.} = \text{Strip width} - b_{F.S.} = 5.0 - 3.5 = 1.50\ m$$

$$\boxed{M.F. = \frac{\text{Field strip العرض الحقيقى لل}}{\text{عرض الشريحة الكلى } \frac{1}{2}} = \frac{1.50}{2.50} = 0.60}$$

$0.45\,M_o = 160.16$ $\qquad$ $0.45\,M_o = 160.16$

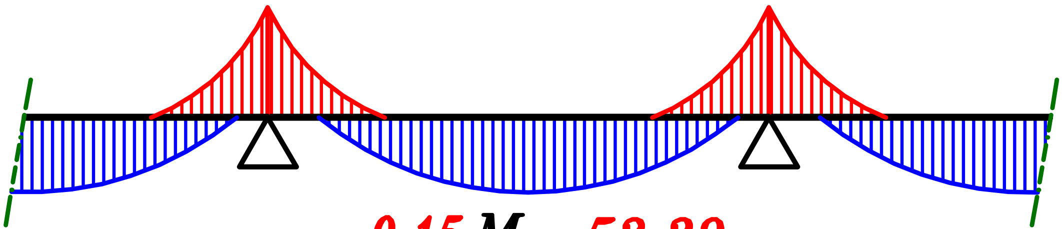
moment C.S.



$0.25\,M_o = 88.98$

$0.15\,M_o = 53.39$ $\qquad$ $0.45\,M_o = 53.39$

moment F.S.



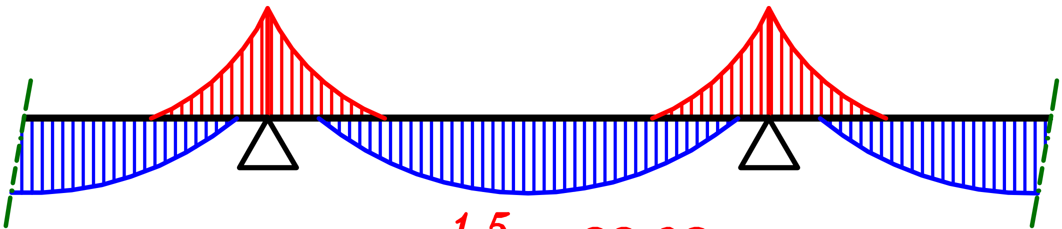
$0.15\,M_o = 53.39$

$$M.F. = \frac{\text{العرض الحقيقى للـ Field strip}}{\text{عرض الشريحه الكلى } \frac{1}{2}} = \frac{1.50}{2.50} = 0.60$$

$53.39 * \frac{1.5}{2.25} = 32.03$ $\qquad$ $53.39 * \frac{1.5}{2.25} = 32.03$

Modified moment F.S.

$b_{F.S.} = 1.50\ m$

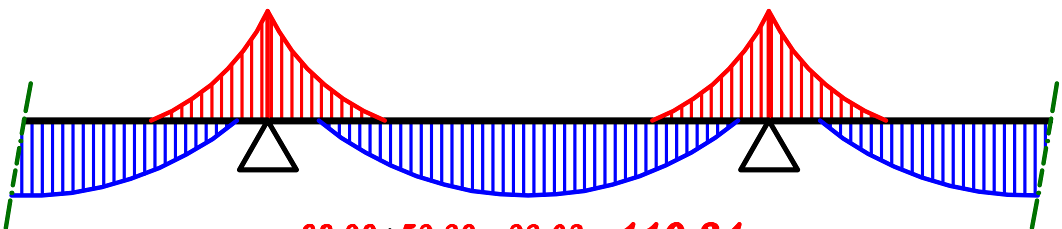


$53.39 * \frac{1.5}{2.25} = 32.03$

$160.16 + 53.39 - 32.03 = 181.52$ $\qquad$ $160.16 + 53.39 - 32.03 = 181.52$

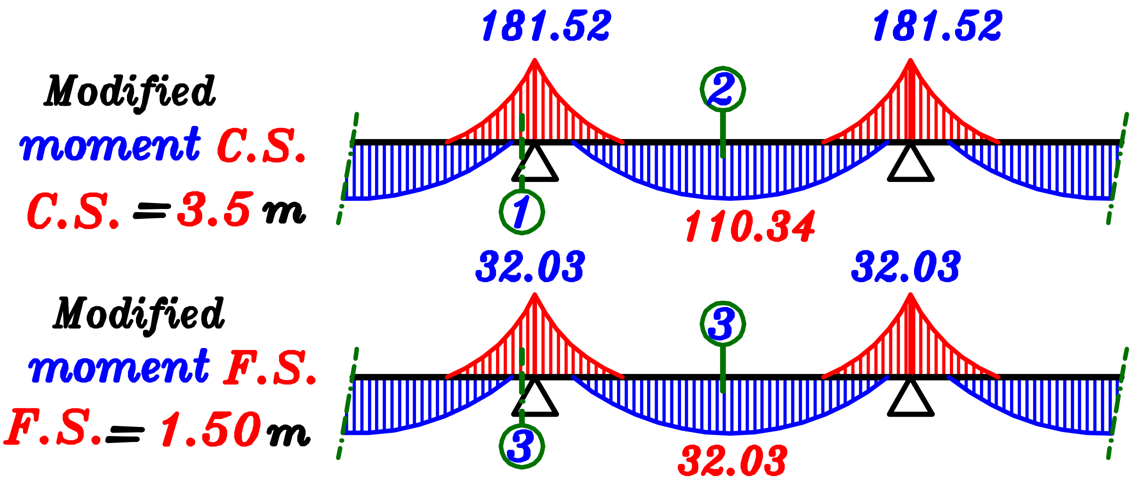
Modified moment C.S.

$b_{c.s.} = 3.5\ m$



$88.98 + 53.39 - 32.03 = 110.34$

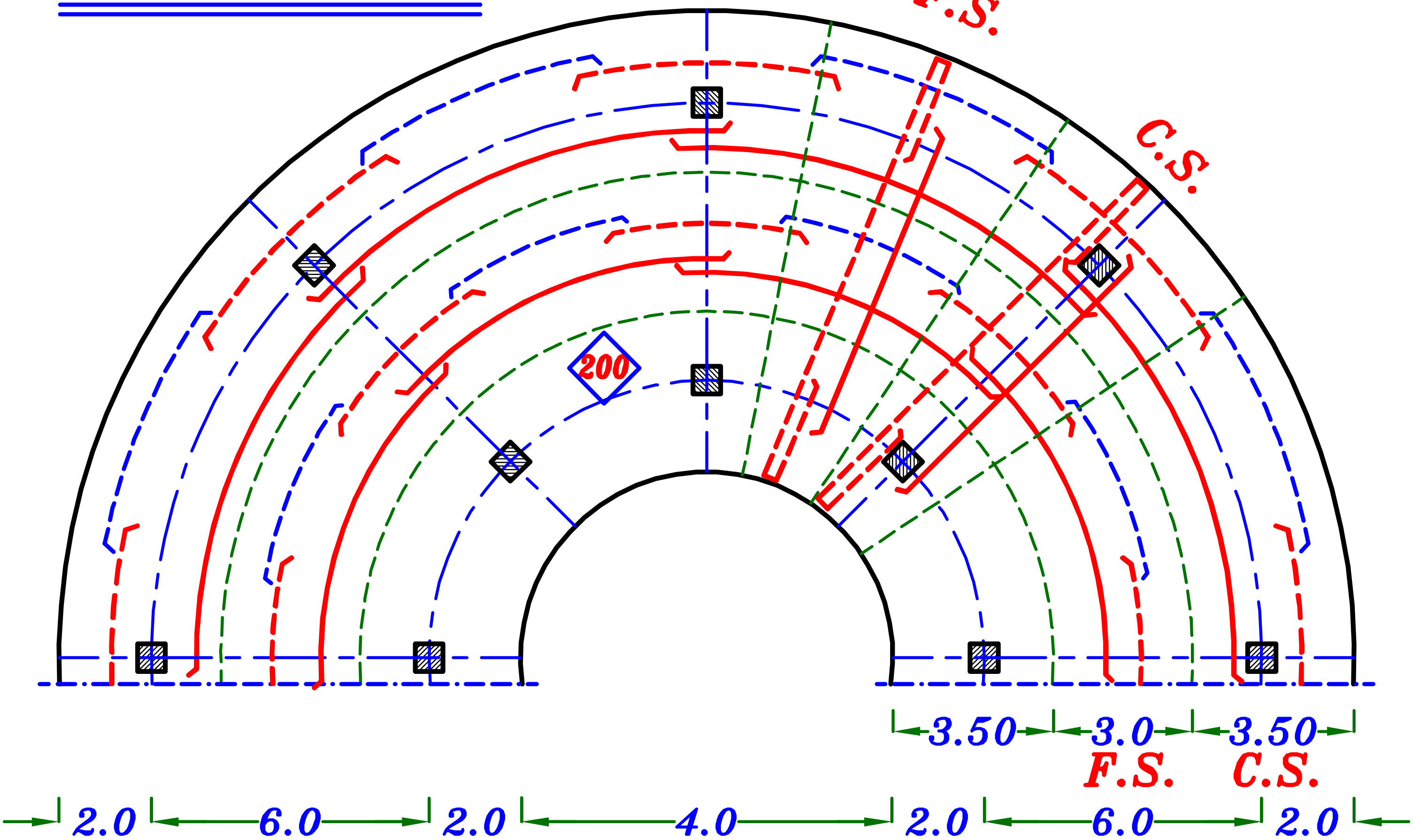
Design the sections of the slab.



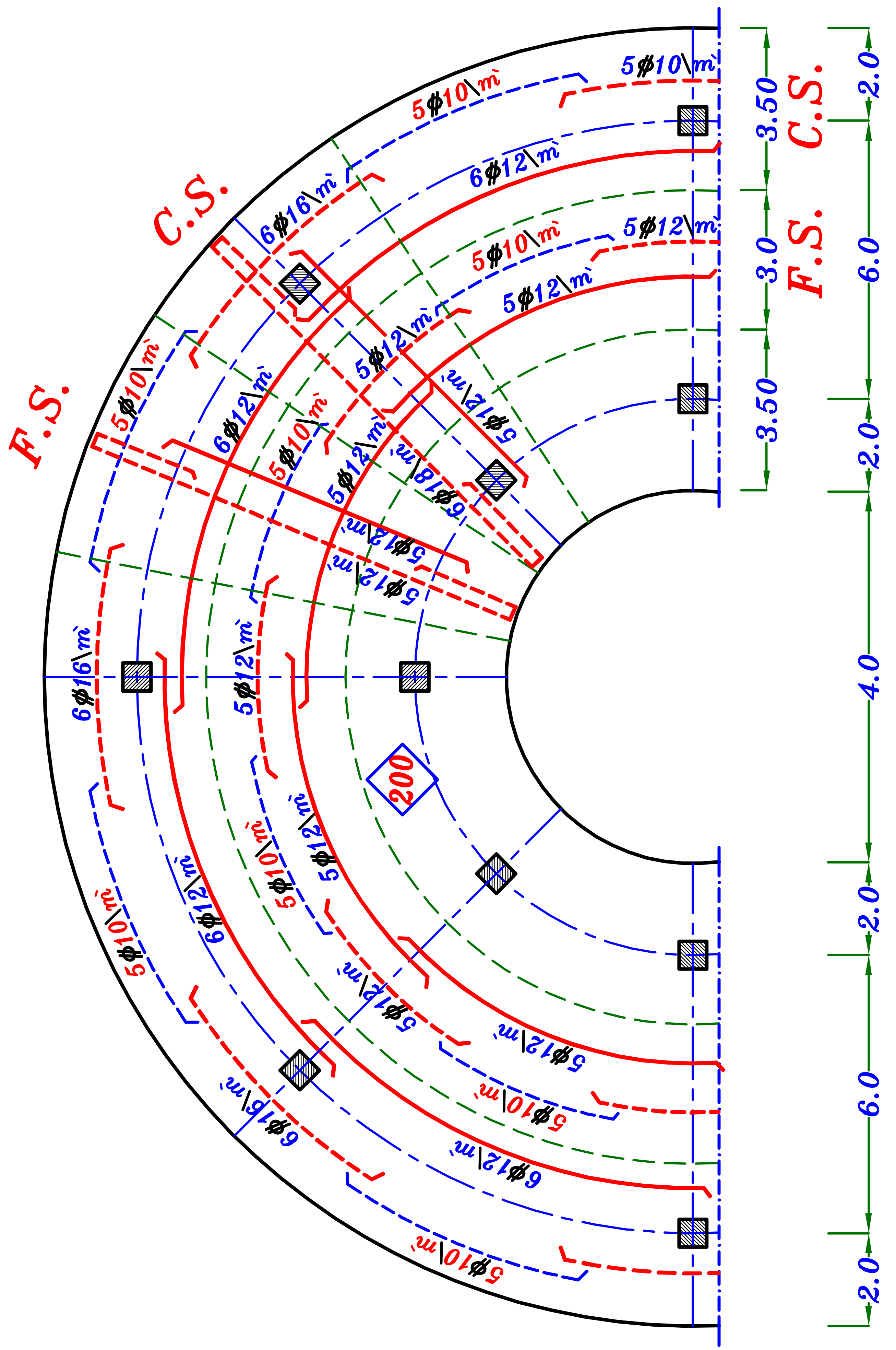
$d = t_s - 30\ mm = 200 - 30 = 170\ mm$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	183.1	3500	170	4.09	0.807	3675	1061	6ϕ16\m
	2	111.3	3500	170	5.24	0.826	2183	628	6ϕ12\m
Field Strip	3	32.3	1500	170	6.34	0.826	633	426	5ϕ12\m

Details of RFT.



Details of RFT.



Example.

The given Figure shows general layout of inclined Flat Slab.

Data.

$F_{cu} = 30 \ N\backslash mm^2$, $F_y = 360 \ N\backslash mm^2$

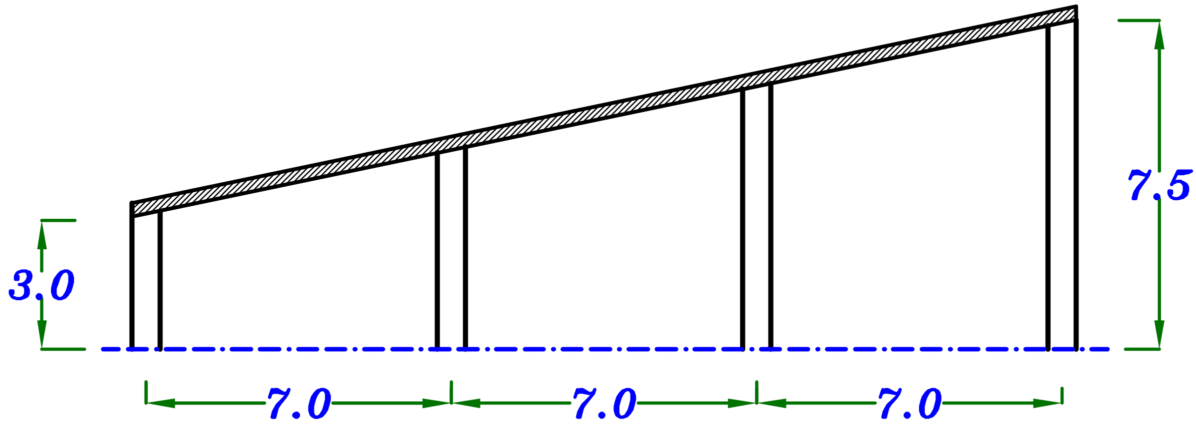
$F.C. = 3.0 \ kN\backslash m^2$, $L.L. = 1.5 \ kN\backslash m^2$ لا توجد حوائط لانه دور أخير

Req.

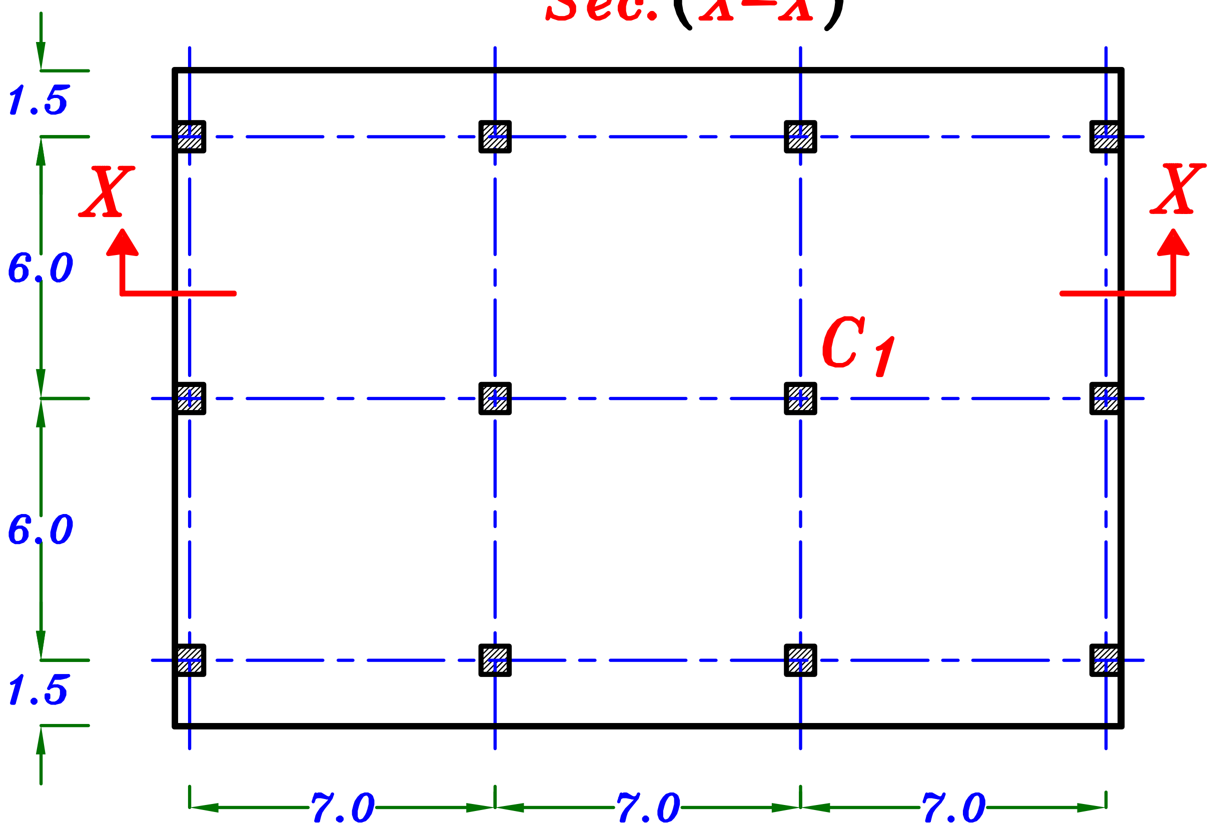
① Check punching on column C_1

② Complete design of typical Floor in both directions.

③ Draw details of reinforcement in plan.

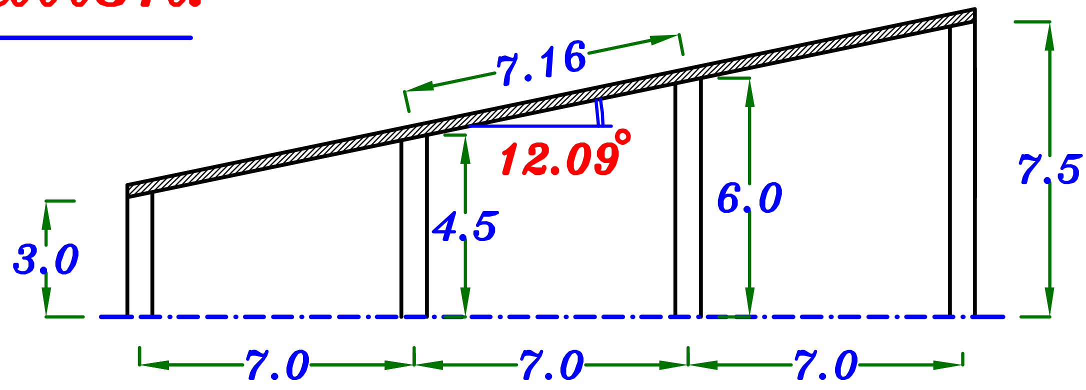


Sec. (X–X)



Plan.

Solution.



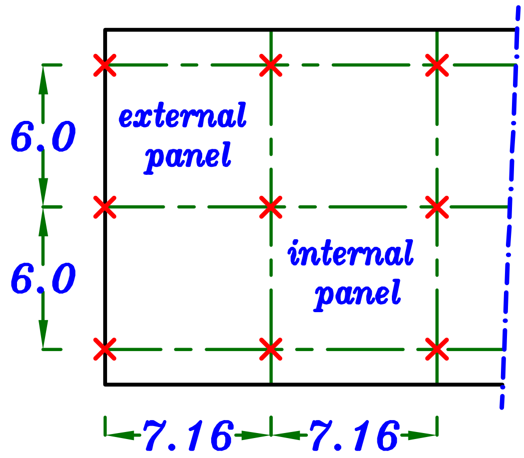
1- Concrete Dimensions.

Column dimensions.

$$b_{Col.} = \begin{cases} 300 \text{ mm} \\ \dfrac{H}{15} = \dfrac{7500}{15} = 500 \text{ mm} \\ \dfrac{L_1}{20} = \dfrac{7160}{20} = 358 \text{ mm} \end{cases}$$

$$\boxed{b_{Col.} = 500 \text{ mm}}$$

(500*500)



Slab Thickness.

$L_1 = 7.16$ m

External panel $t_s = \dfrac{L_1}{32} = \dfrac{7160}{32} = 223.7$ mm

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7160}{36} = 198.9$ mm

$$\boxed{t_s = 240 \text{ mm}}$$

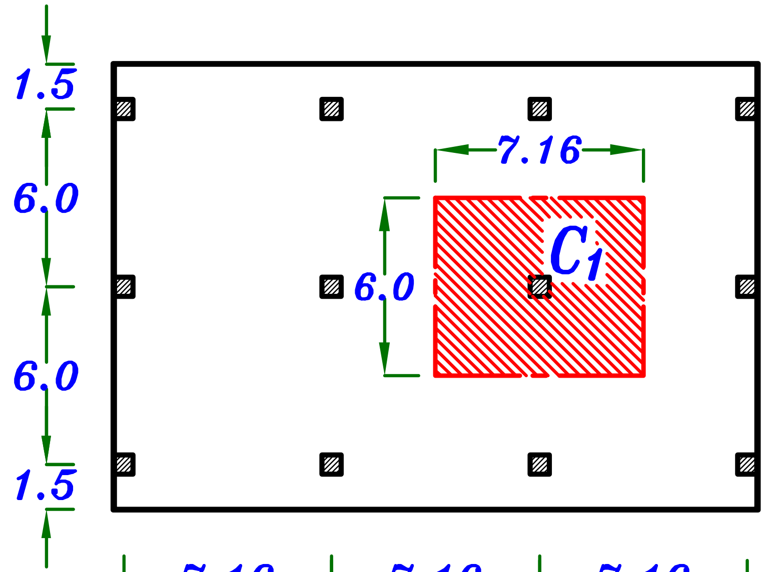
2- Loads on the Slab.

$$w_{si} = 1.4\,(t_s\,\gamma_c + F.C.) + 1.6\,(L.L.)\cos\theta$$

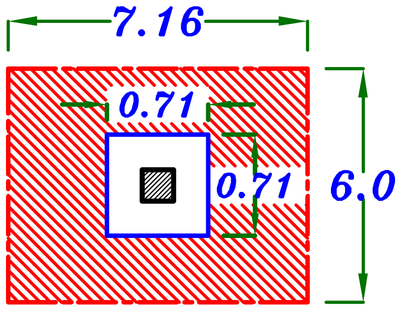
$$w_{si} = 1.4\,(0.24 * 25 + 3.0) + 1.6\,(1.5)\cos 12.09^\circ = 14.94 \text{ kN}\backslash\text{m}^2$$

3 – Check Punching on interior column C_1

كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى



C_1 Interior Column.



$$d = t_s - 30\,mm = 240 - 30 = 210\ mm = 0.21\ m$$

$$C + d = 0.50 + 0.21 = 0.71\ m$$

$$Q_{pu} = w_s\left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 14.94\left[7.16 * 6.0 - 0.71 * 0.71\right] = 634.3\ kN$$

$$A_p = (b_o * d) = (4 * 710) * 210 = 596400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{634.3 * 10^3}{596400} * 1.15 = 1.22\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$$

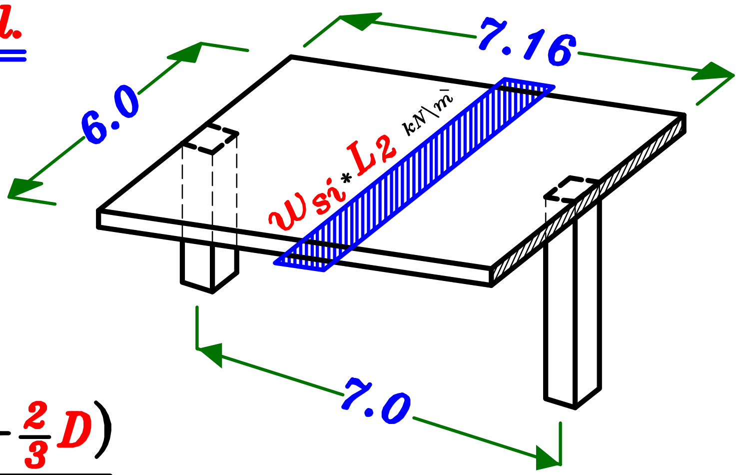
$$\boxed{q_{pu} < q_{pcu}} \longrightarrow \text{Safe Punching.}$$

Moment at Long Direction.

Use Empirical Method.

فى الاتجاه الطويل نختار حساب العزوم بطريقه **Empirical Method** لانها أسهل .
و خاصه أن عدد البواكي فى هذا الاتجاه ٣ بواكي .

Total moment on the panel.



Span $= 7.16$ m

Width $= 6.0$ m

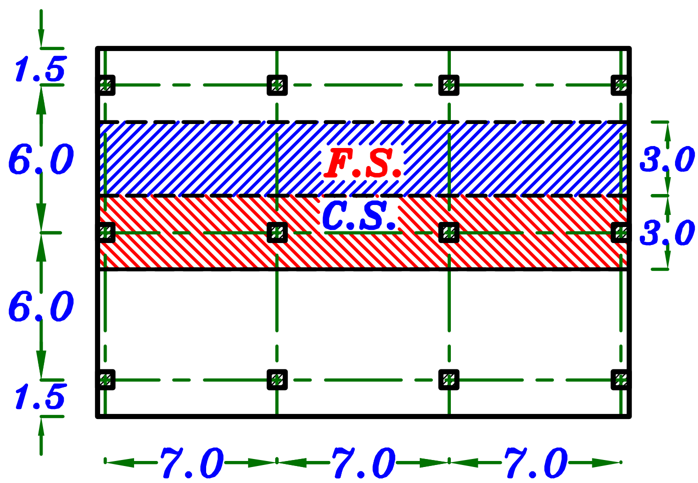
$$M_o = \frac{w L L^`}{8}$$

$$M_o = \frac{(w_s * L_2)\left(L_1 - \frac{2}{3}D\right)\left(L_1^` - \frac{2}{3}D\right)}{8}$$

$$M_o = \frac{(14.94 * 6.0)\left(7.0 - \frac{2}{3} * 0.50\right)\left(7.16 - \frac{2}{3} * 0.50\right)}{8}$$

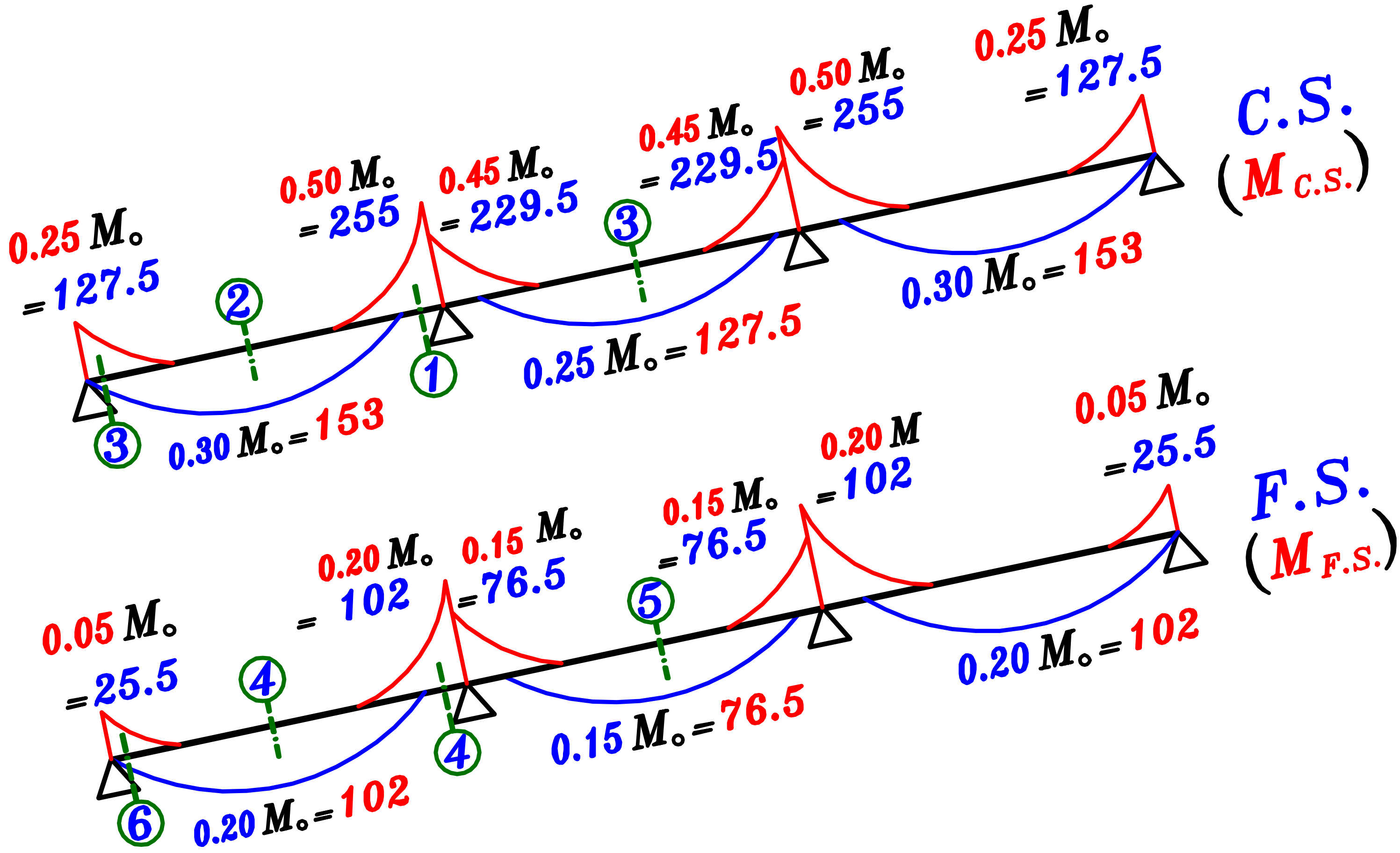
$$\boxed{M_o = 509.95 \text{ kN.m}}$$ Long Direction

4- Distribute the B.M. (M_o) on C.S. & F.S.



Column Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0$ m

Field Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0$ m



Design of sections.

$$d = t_s - 30\ mm = 240 - 30 = 210\ mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	A_s (mm²/b)	A_s (mm²/m)	No. of bars/m
Column Strip	1	255.0	3000	210	3.94	0.801	4211	1403	7ϕ16\m
	2	153.0	3000	210	5.09	0.826	2450	816	8ϕ12\m
	3	127.5	3000	210	5.58	0.826	2041	680	7ϕ12\m
Field Strip	4	102.0	3000	210	6.23	0.826	1633	544	5ϕ12\m
	5	76.5	3000	210	7.20	0.826	1225	408	5ϕ12\m
	6	25.5	3000	210	12.4	0.826	408	136	5ϕ12\m

Moment at Short Direction.

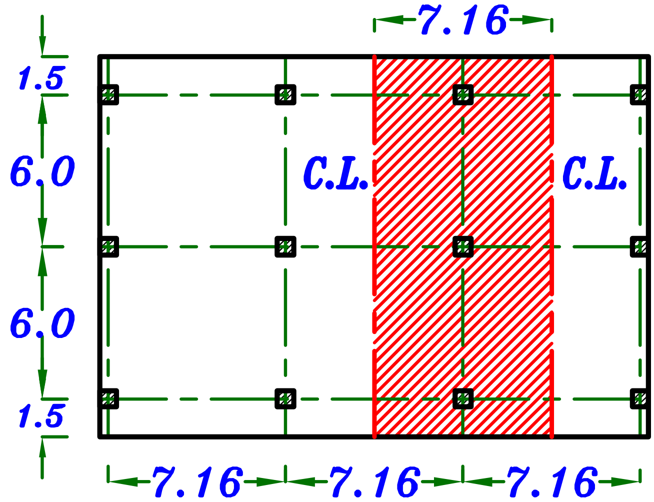
Use Frame analysis Method.

$Span = L_2 = 6.0\ m$

$Width = L_1 = 7.16\ m$

$b_{C.S.} = \dfrac{L_2}{2} = 3.0\ m$

$b_{F.S.} = L_1 - \dfrac{L_2}{2} = 4.16\ m$

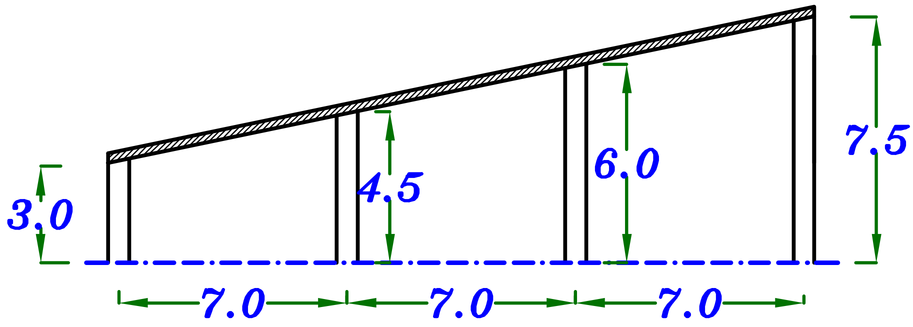


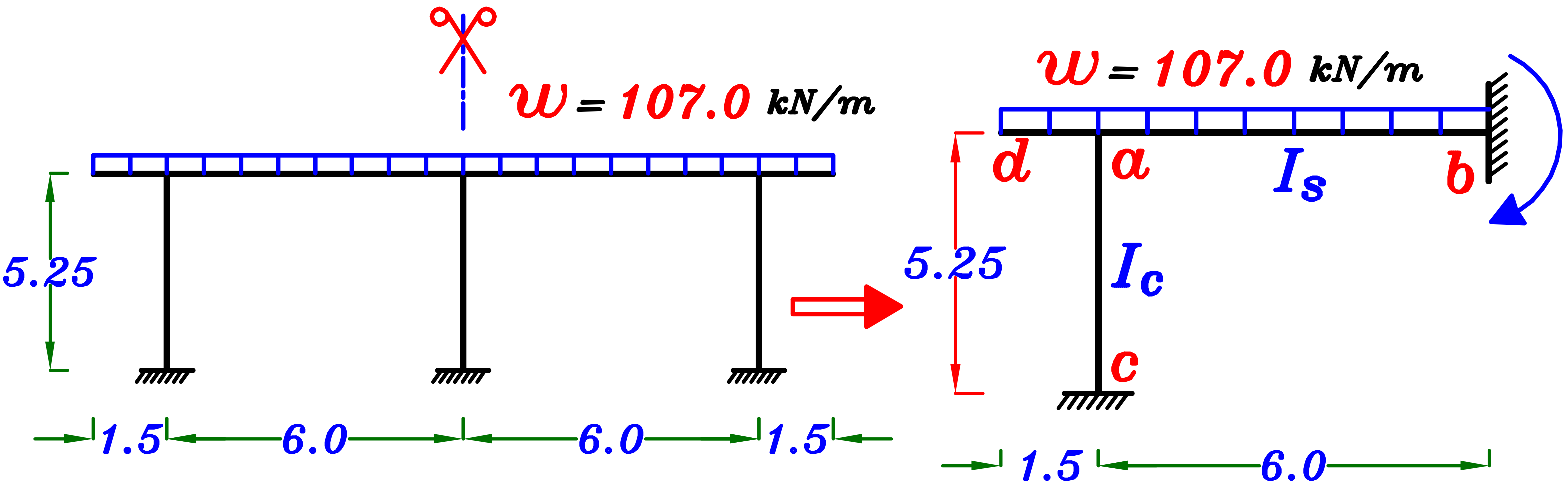
$w = w_s * L_2 = 14.94 * 7.16 = 107.0\ kN/m$

نأخذ ارتفاع عمود ال **Frame**

بمتوسط ارتفاع الاعمده كلها

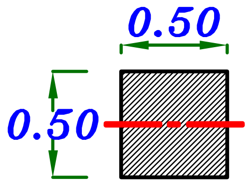
$$\boxed{h = \frac{3.0 + 7.5}{2.0} = 5.25\ m}$$



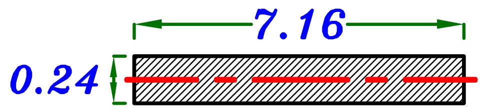


ⓐ **Calculate Moment of Inertia For Slabs & Columns.**

عمود خارجي



$$I_c = 0.6 * \frac{b(t)^3}{12} = 0.6 * \frac{0.50 * 0.50^3}{12} = 3.125 * 10^{-3}\ m^4$$



$$I_s = \frac{L_1 * t_s^3}{12} = \frac{7.16 * 0.24^3}{12} = 8.25 * 10^{-3}\ m^4$$

ⓑ **Calculate the stiffness For each member.**

$$K_{ab} = \frac{I_s}{L} = \frac{8.25 * 10^{-3}}{6.0} = 1.375 * 10^{-3}$$

$$K_{ac} = \frac{I_c}{h} = \frac{3.125 * 10^{-3}}{5.25} = 5.95 * 10^{-4}$$

ⓒ **Calculate the Distribution Factors. (D.F.)**

For Joint a

$$\Sigma K = K_{ab} + K_{ac} = 1.375 * 10^{-3} + 5.95 * 10^{-4} = 1.97 * 10^{-3}$$

$$D.F.(ab) = \frac{K_{ab}}{\Sigma K} = \frac{1.375 * 10^{-3}}{1.97 * 10^{-3}} = 0.70$$

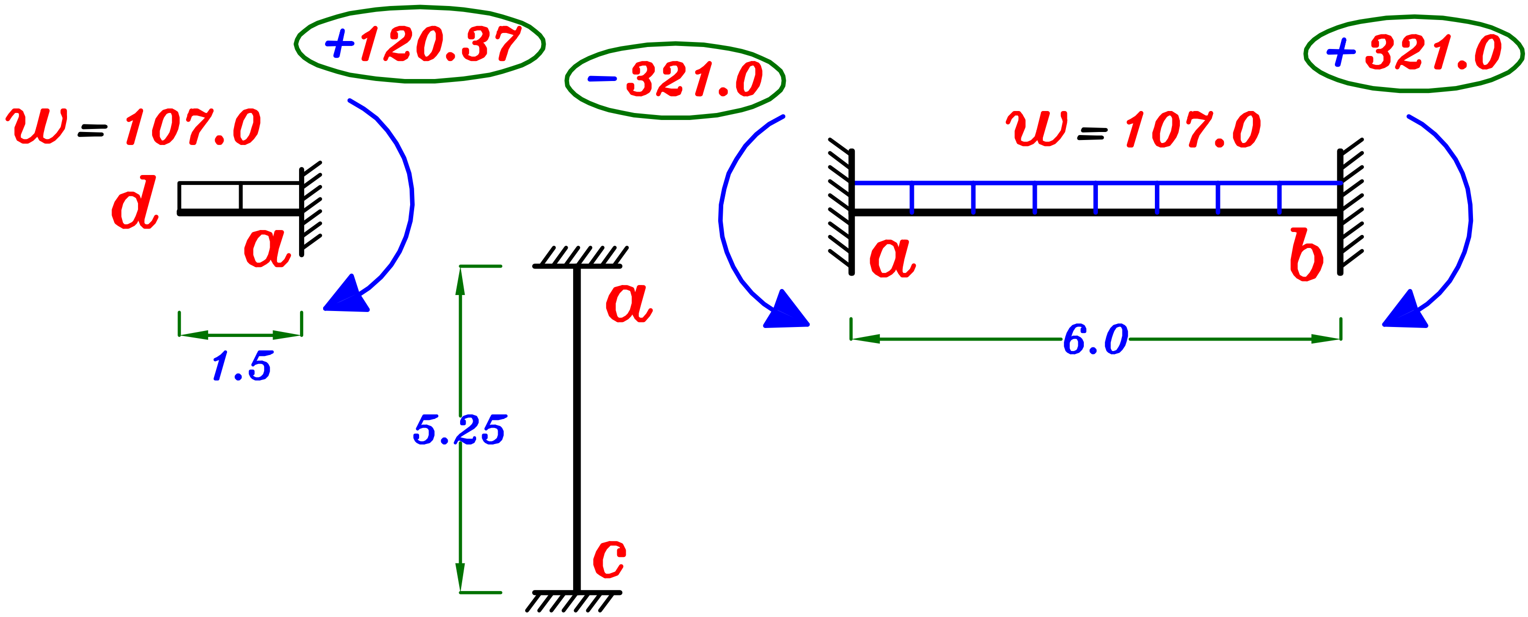
$$D.F.(ac) = \frac{K_{ac}}{\Sigma K} = \frac{5.95 * 10^{-4}}{1.97 * 10^{-3}} = 0.30$$

ⓓ **Calculate Fixed End Moment For the Slab.**

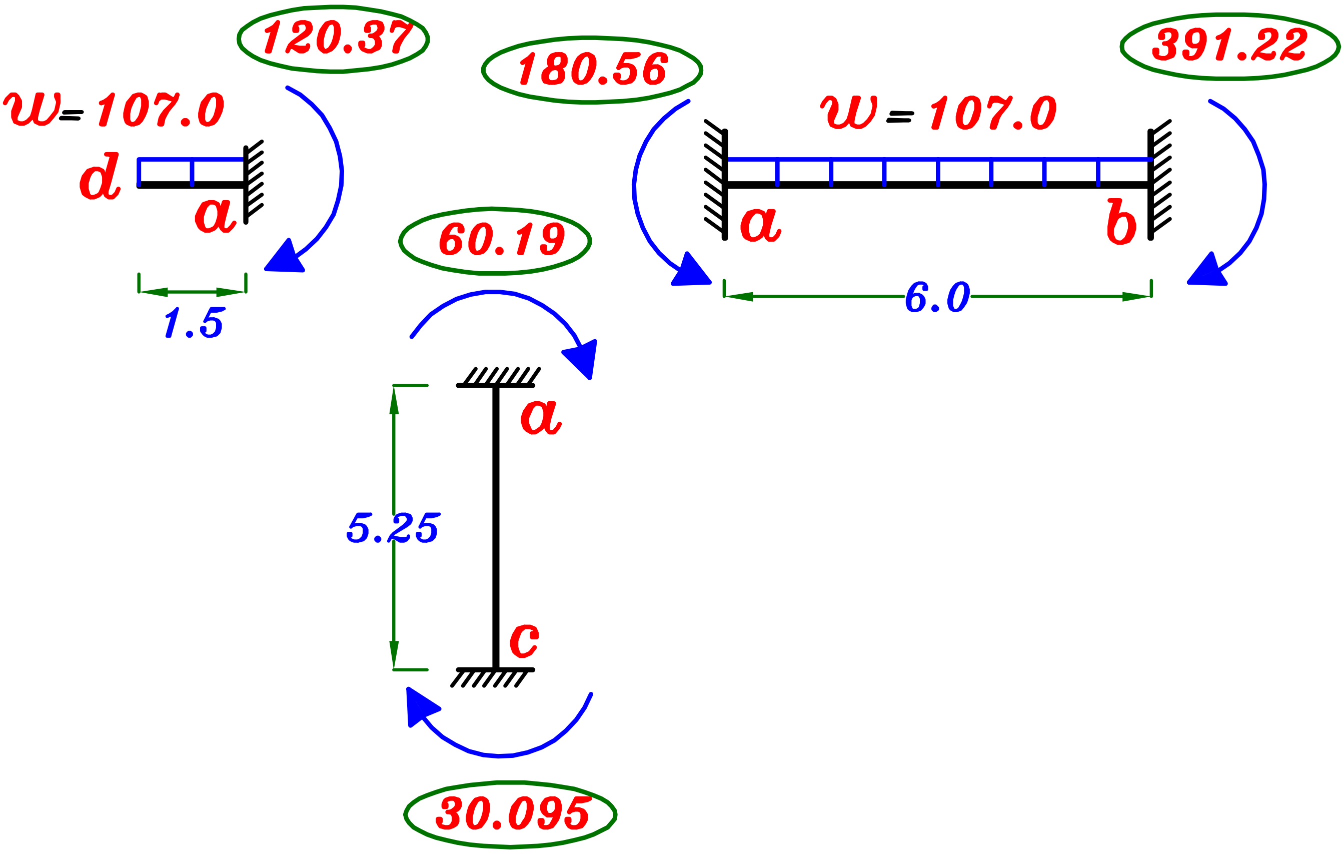
$$F.E.M.(ab) = -\frac{wL^2}{12} = -\frac{107.0 * 6.0^2}{12} = -321.0\ kN.m.$$

$$F.E.M.(ba) = +\frac{wL^2}{12} = +\frac{107.0 * 6.0^2}{12} = +321.0\ kN.m.$$

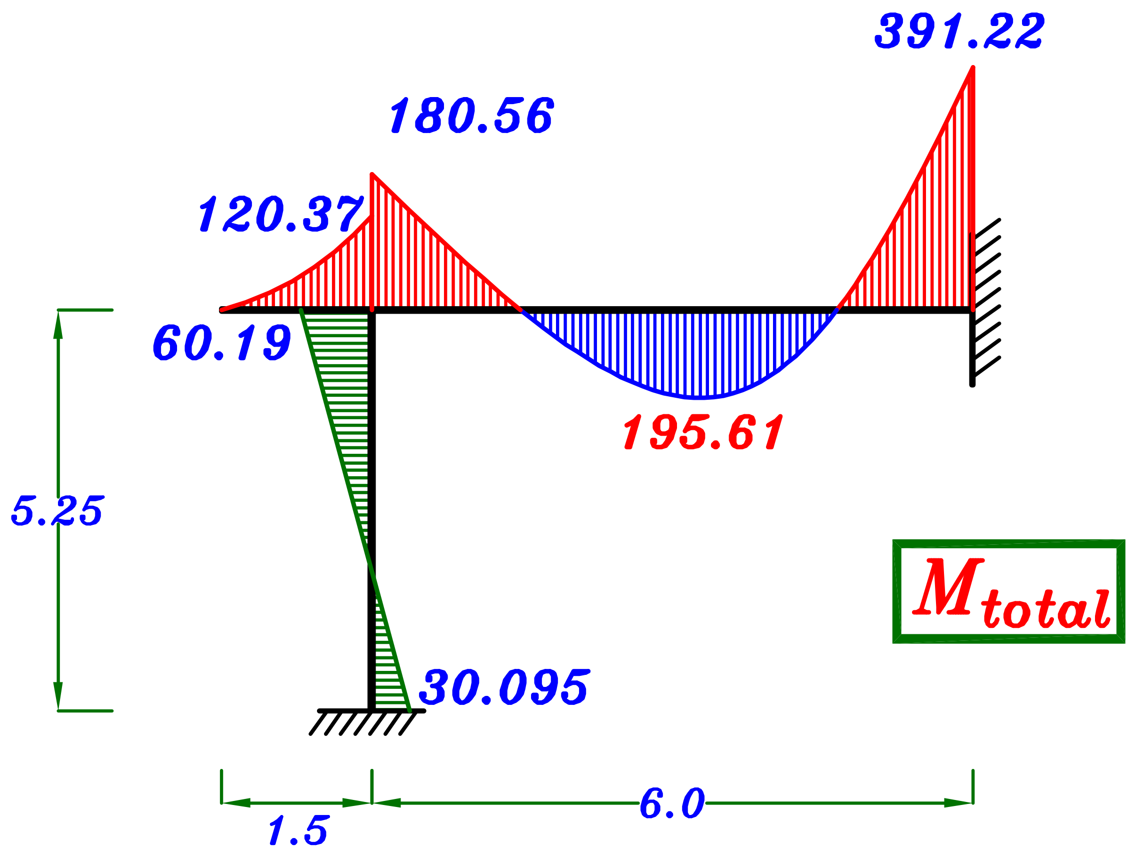
$$F.E.M.(ac) = +\frac{wL^2}{2} = +\frac{107.0 * 1.5^2}{2} = +120.37\ kN.m.$$



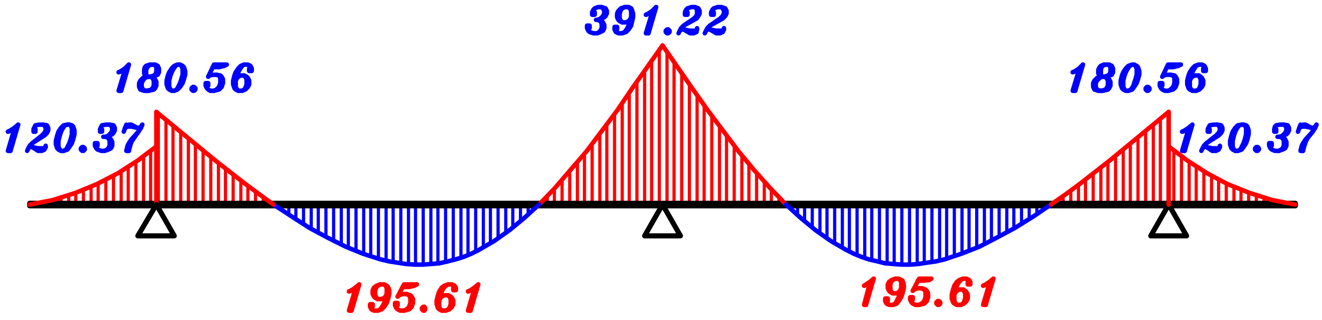
Joint	c	a			b
member	c − a	a − c	a − d	a − b	b − a
D.F.	0	0.30	0	0.70	0
F.E.M.	0	0	+120.37	−321.0	+321.0
B.M.	0	+60.19	0	+140.44	0
C.O.M.	+30.095	0	0	0	+70.22
B.M.	0	0	0	0	0
M_F	+30.095	+60.19	+120.37	−180.56	+391.22



120.37
w= 107.0
d
a
1.5
180.56
391.22
w = 107.0
a
b
6.0
60.19
a
5.25
c
30.095



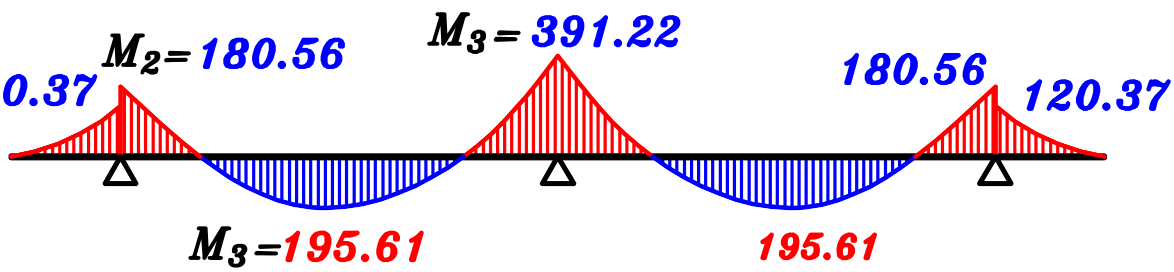
391.22
180.56
120.37
60.19
195.61
5.25
M_{total}
30.095
1.5
6.0



391.22
180.56
180.56
120.37
120.37
195.61
195.61

Distribute the moment of the Frame on Column Strip and Field Strip.

M_{total}

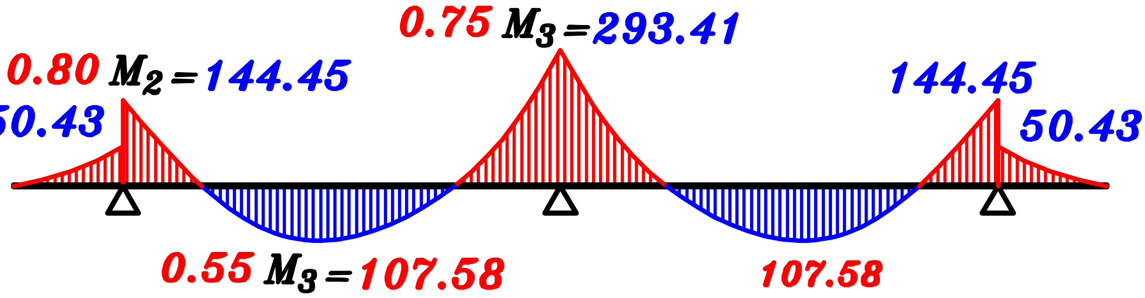


عرض ال C.S.

$M_1 * \frac{3.0}{7.16} = 50.43$

عرض الشريحة كله

moment C.S.

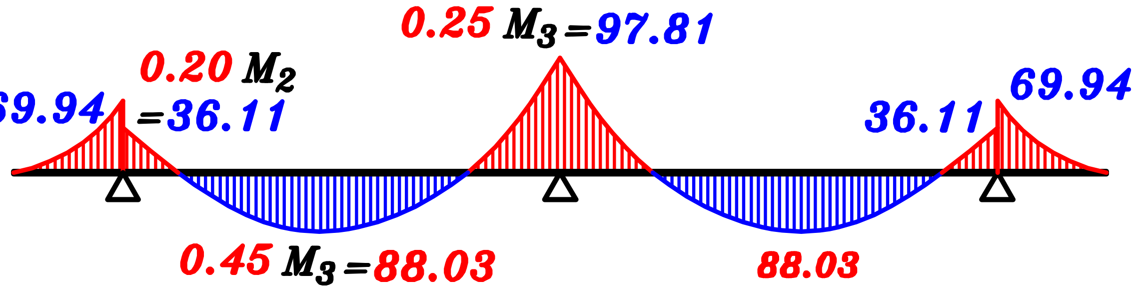


عرض ال F.S.

$M_1 * \frac{4.16}{7.16} = 69.94$

عرض الشريحة كله

moment F.S.

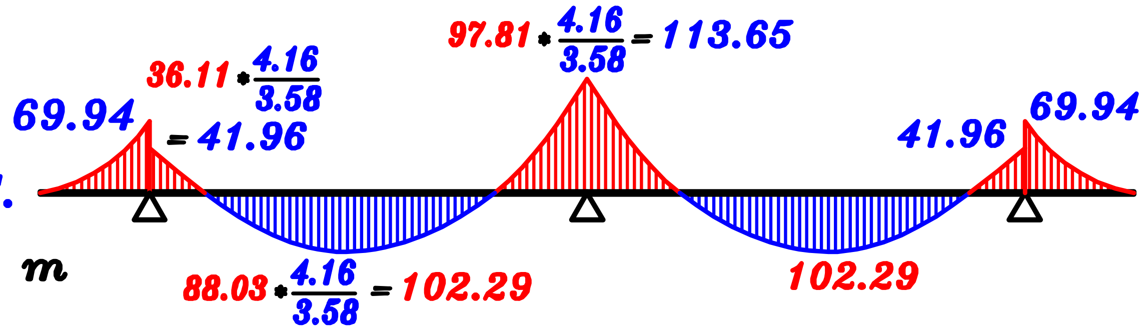


$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{4.16}{3.58}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.16}{3.58}}$$

Modified moment F.S.

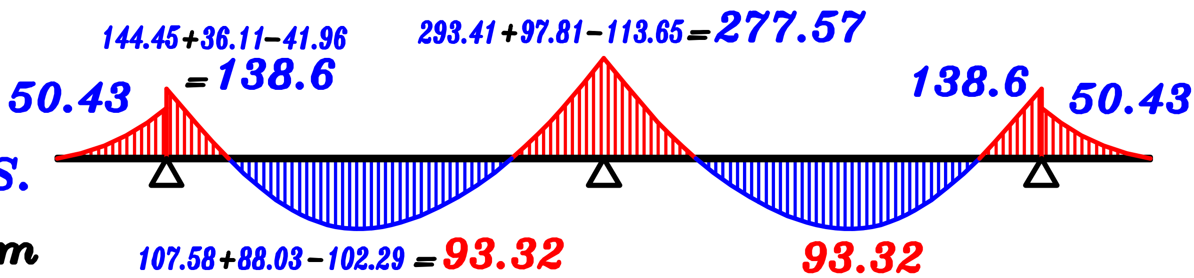
$b_{F.S.} = 4.16\ m$



$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

Modified moment C.S.

$b_{C.S.} = 3.0\ m$

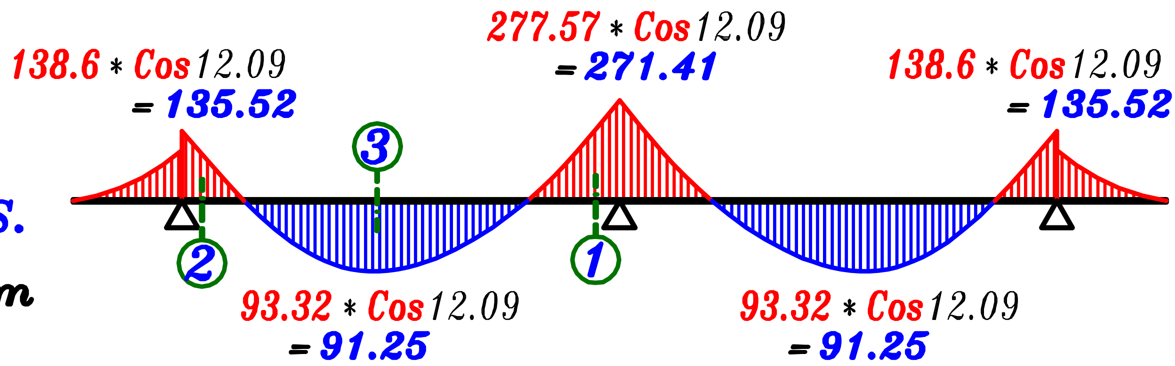


Design of sections.

شريحه أفقيه فى بلاطه مائله اذا تصمم دائماً على $(M * \cos\theta)$

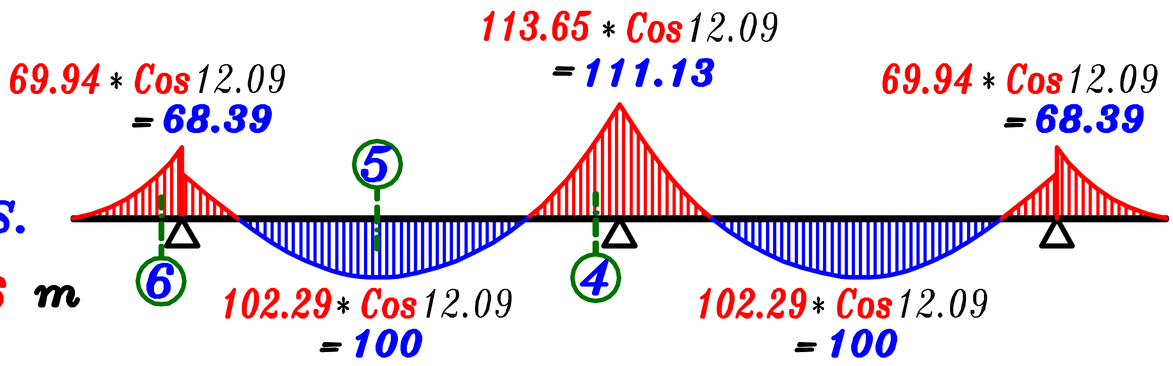
Modified moment C.S.

$b_{C.S.} = 3.0\ m$



Modified moment F.S.

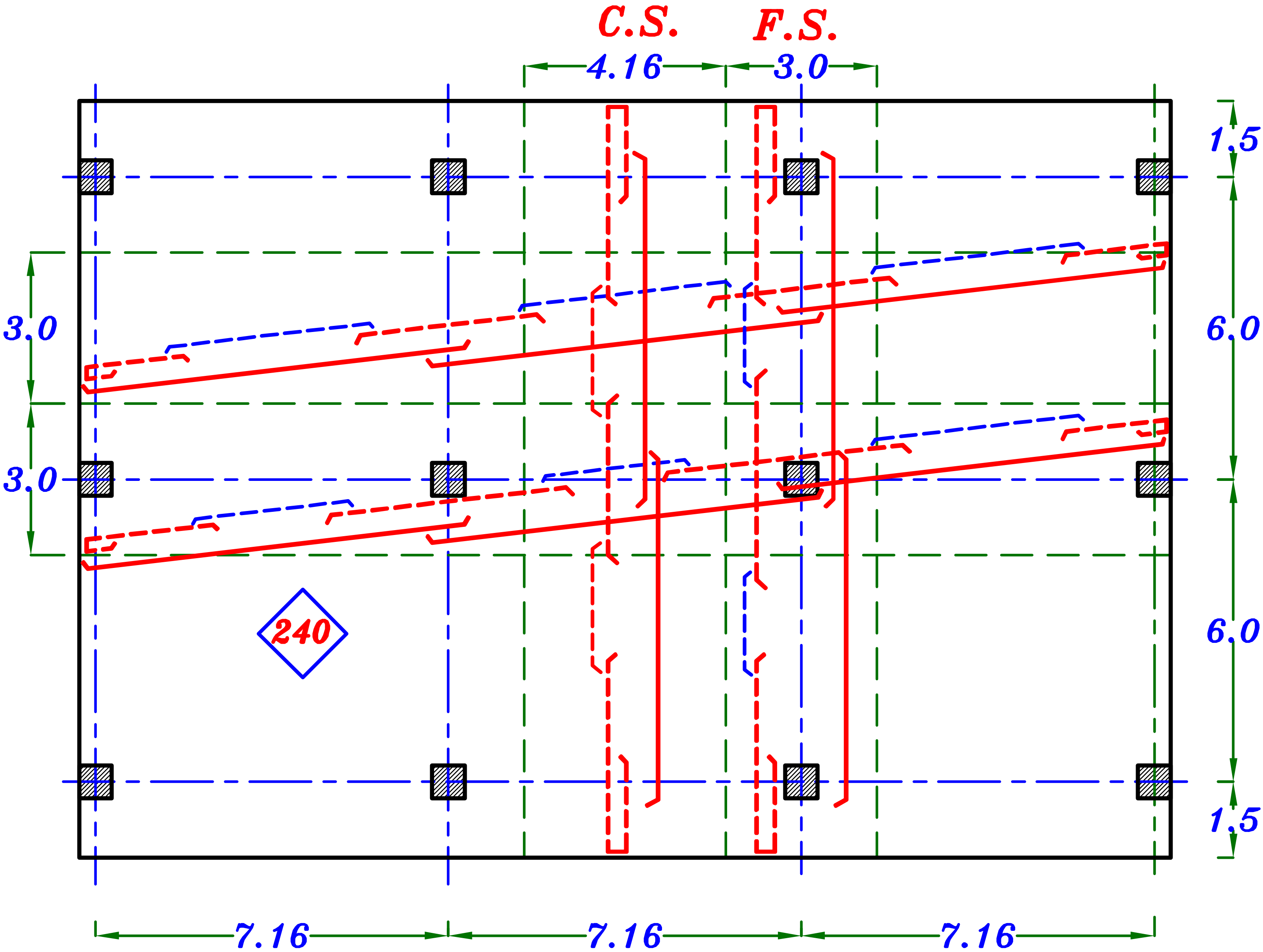
$b_{F.S.} = 4.16\ m$



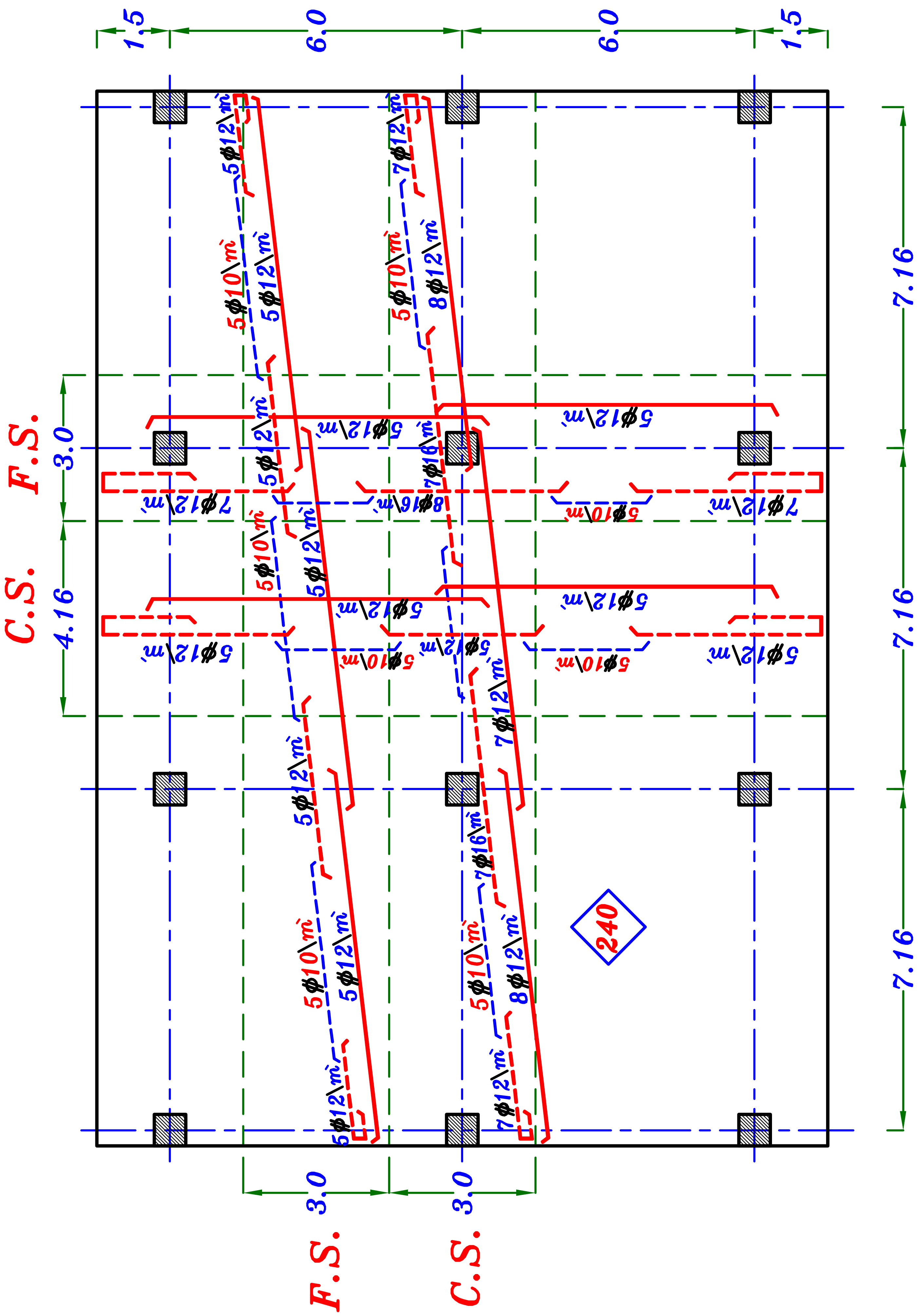
$$d = t_s - 40\ mm = 240 - 40 = 200\ mm$$

Strip	Sec.	M (kN.m/strip)	b (m)	d (mm)	C_1	J	As (mm^2/b)	As (mm^2/m)	No. of bars/m
Column Strip	1	271.41	3000	200	3.64	0.787	4790	1596	8ϕ16\m
	2	135.52	3000	200	5.15	0.826	2278	759	7ϕ12\m
	3	91.25	3000	200	6.28	0.826	1534	511	5ϕ12\m
Field Strip	4	111.13	4160	200	6.70	0.826	1868	449	5ϕ12\m
	5	100.0	4160	200	7.06	0.826	1681	404	5ϕ12\m
	6	68.39	4160	200	8.54	0.826	1150	276	5ϕ12\m

Details of RFT.



Details of RFT.



Example.

The given plan shows general layout of a Flat slab Floor

The column height 4.5 m

The building consist of Ground Floor and Five typical Floors.

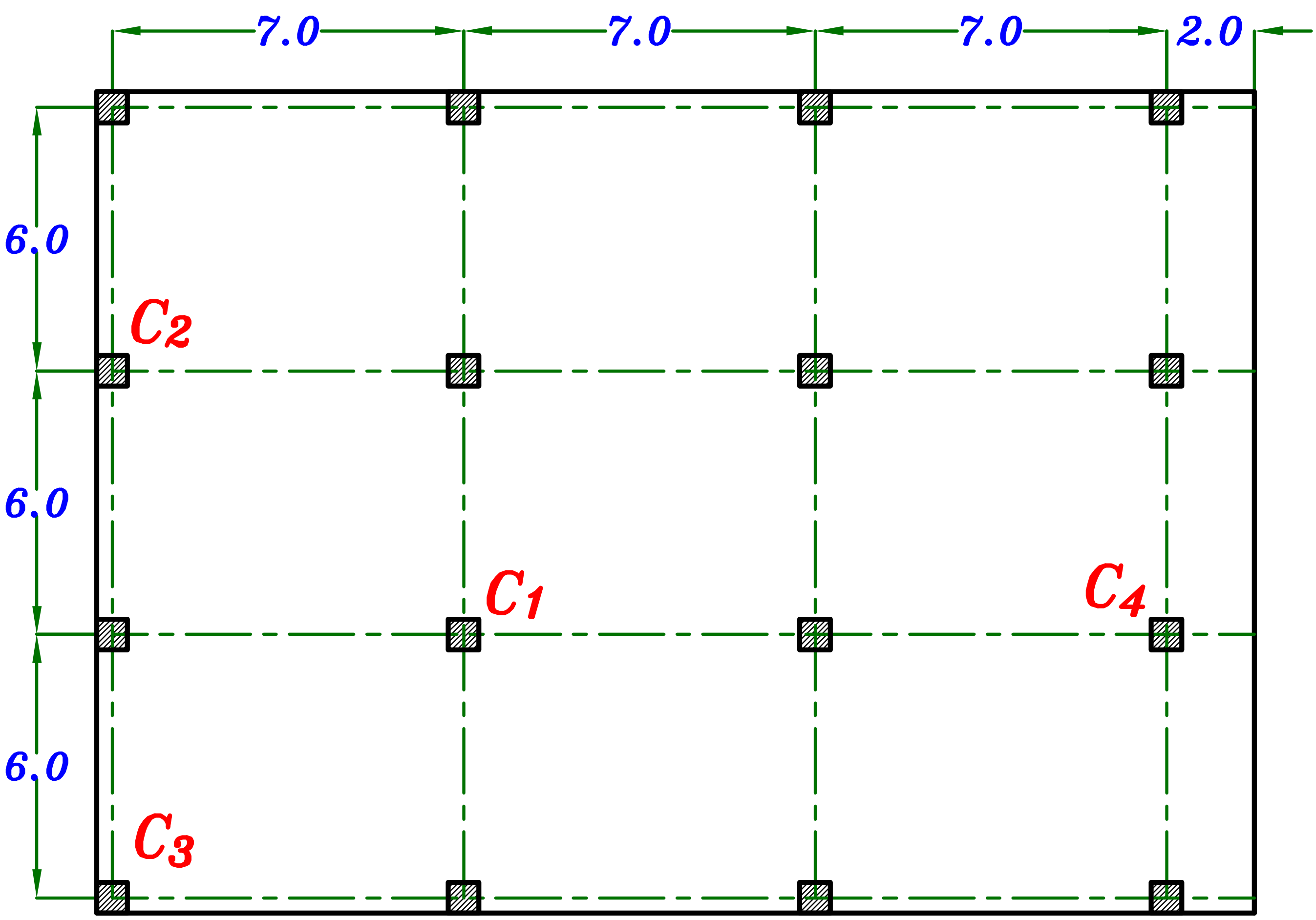
Data.

$F_{cu} = 30\ N\backslash mm^2 \quad F_y = 360\ N\backslash mm^2$

$F.C. = 0.8\ kN\backslash m^2$, $L.L. = 2.0\ kN\backslash m^2$, $Walls = 1.5\ kN\backslash m^2$

Req.

1. Check punching on column C_1
2. Design the columns C_1 , C_2 , C_3 & C_4 at ground Floor & Last Floor.



Solution.

Concrete Dimensions.

Column dimensions.

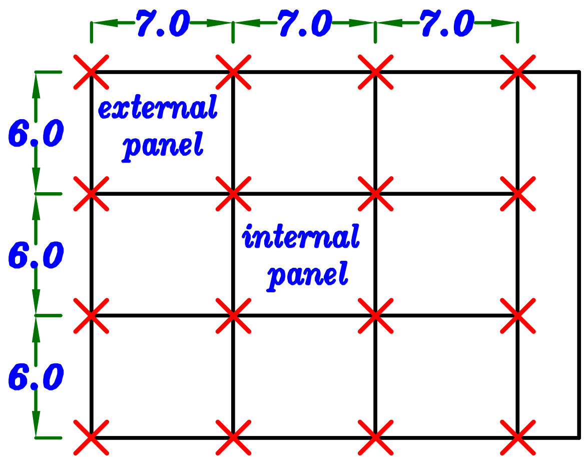
$$b_{Col.} = \begin{cases} 300\ mm \\ \dfrac{H}{15} = \dfrac{4500}{15} = 300\ mm \\ \dfrac{L_1}{20} = \dfrac{7000}{20} = 350\ mm \end{cases}$$

$$\boxed{b_{Col.} = 400\ mm}$$

$(400 * 400)$

Slab Thikness.

$$L_1 = 8.0\ m$$



External panel $t_s = \dfrac{L_1}{32} = \dfrac{7000}{32} = 218.7\ mm$

Internal panel $t_s = \dfrac{L_1}{36} = \dfrac{7000}{36} = 194.4\ mm$

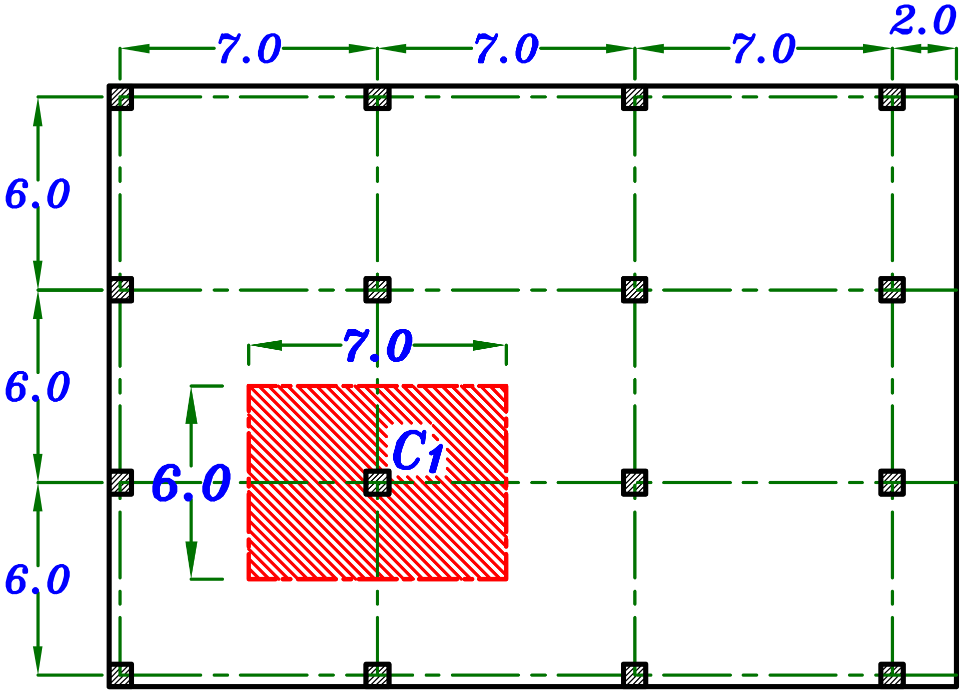
$$\boxed{t_s = 220\ mm}$$

Loads on the Slab.

$$w_s = 1.4\,(t_s\,\gamma_c + F.C. + Walls) + 1.6\,(L.L.)$$

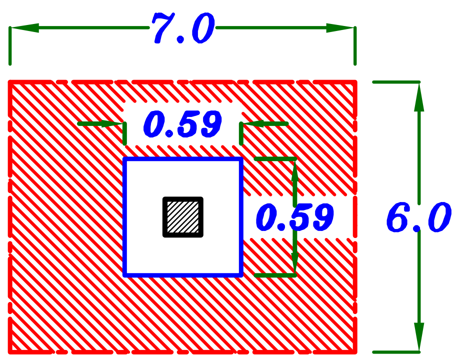
$$w_s = 1.4\,(0.22 * 25 + 0.8 + 1.5) + 1.6\,(2.0) = 14.2\ kN\backslash m^2$$

1 – Check punching on column C_1



كل عمود يحمل مساحه
من C.L. البلاطه
الى C.L. البلاطه الاخرى

C_1 Interior Column.



$$d = t_s - 30\,mm = 220 - 30 = 190\ mm = 0.19\ m$$

$$C + d = 0.40 + 0.19 = 0.59\ m$$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 14.2 \left[7.0 * 6.0 - 0.59 * 0.59\right] = 591.4\ kN$$

$$A_p = (b_o * d) = (4 * 590) * 190 = 448400\ mm^2$$

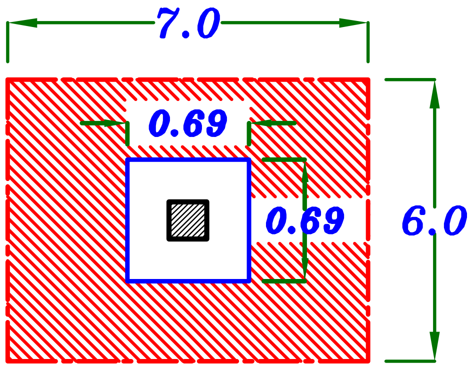
$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{591.4 * 10^3}{448400} * 1.15 = 1.51\ N/mm^2$$

$$q_{pcu} = 0.316\sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316\sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$$

$$\boxed{q_{pu} > q_{pcu}}$$

→ Unsafe punching
Increase dimensions of the column

C_1 Interior Column.



Take the Column (500 * 500)

$d = t_s - 30\ mm = 220 - 30 = 190\ mm = 0.19\ m$

$C + d = 0.50 + 0.19 = 0.69\ m$

$$Q_{pu} = w_s \left[L_1 * L_2 - (C_1 + d)(C_2 + d)\right]$$

$$Q_{pu} = 14.2 \left[7.0 * 6.0 - 0.69 * 0.69\right] = 589.6\ kN$$

$$A_p = (b_o * d) = (4 * 690) * 190 = 524400\ mm^2$$

$$q_{pu} = \frac{Q_{pu}}{A_p} * \beta = \frac{589.6 * 10^3}{524400} * 1.15 = 1.29\ N/mm^2$$

$$q_{pcu} = 0.316 \sqrt{\frac{F_{cu}}{\gamma_c}} = 0.316 \sqrt{\frac{30}{1.5}} = 1.41\ N/mm^2$$

$\boxed{q_{pu} < q_{pcu}} \longrightarrow$ Safe Punching.

عند تصميم العمود C_1 يجب أن لا تقل أبعاد العمود عن (500 * 500)
حتى تكون البلاطه Safe Punching.

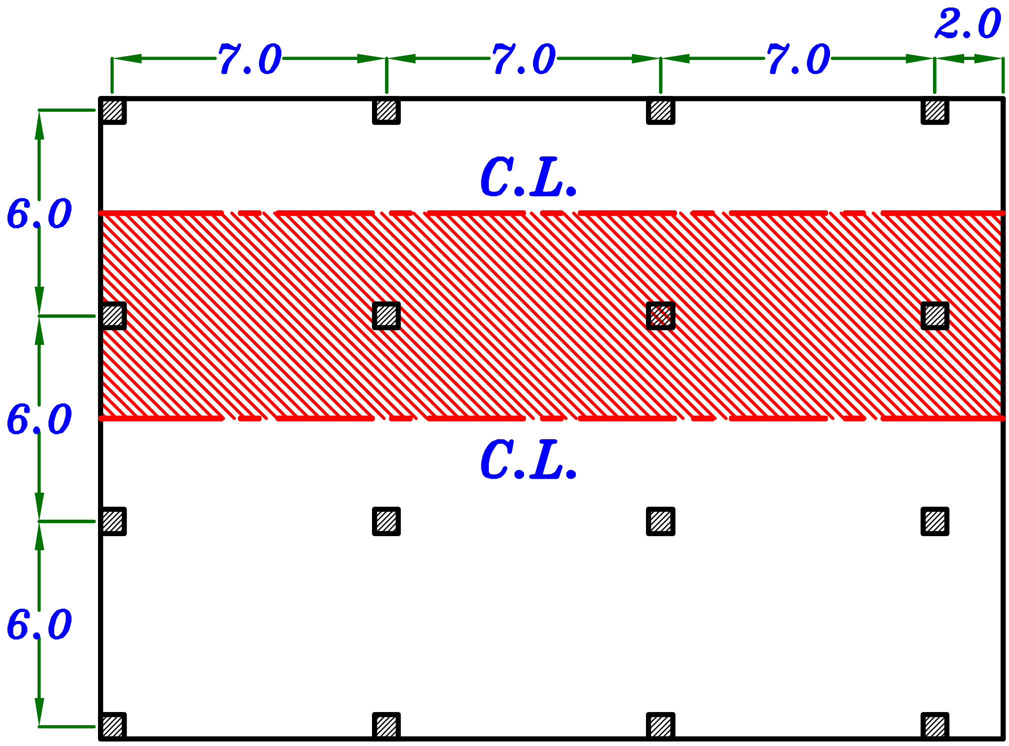
Calculations of moments on Flat slab.

Take a Strips in the slabs at the long and short directions.
The strip width From C.L. the slab to C.L. the slab.
and Calculate the moment on the panel.

Strip at Long Direction.

Span $= 7.0\ m$

Width $= 6.0\ m$



$$M_o = \frac{(w_s * L_2)\,(L_1 - \frac{2}{3}D)^2}{8} = \frac{(14.20 * 6.0)\,(7.0 - \frac{2}{3} * 0.5)^2}{8}$$

$$\boxed{M_o = 473.3\ kN.m}$$ Long Direction

Cantilever Moment.

$$M_{Cant.}\backslash \grave{m} = \frac{w_s * (L_c)^2}{2} = \frac{14.20 * (2.0)^2}{2} = 28.4\ kN.m/m$$

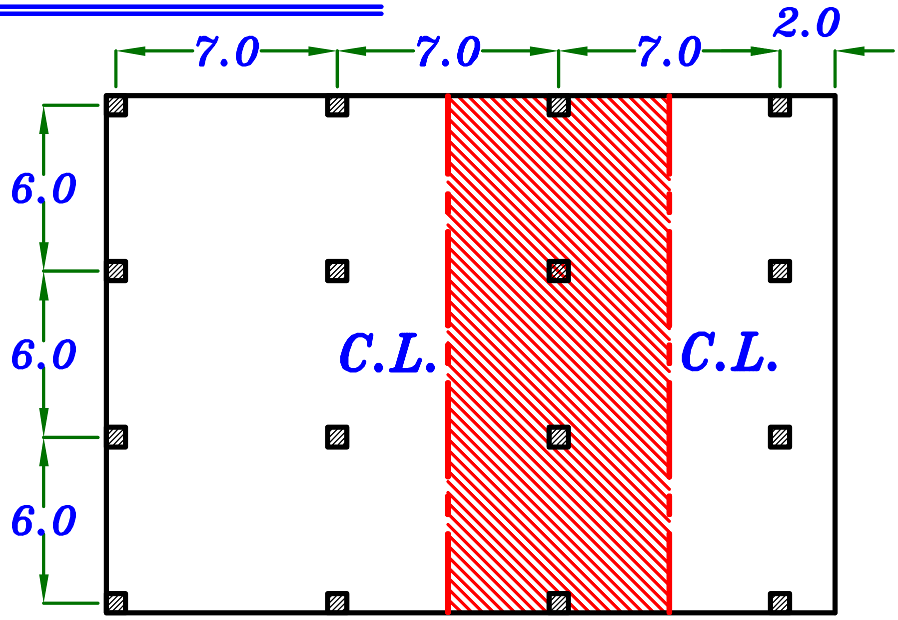
$$M_{Cant.}(C.S.) = M_{Cant.}\backslash \grave{m} * b_{C.S.} = 28.4 * 3.0 = 85.20\ kN.m$$

$$M_{Cant.}(F.S.) = M_{Cant.}\backslash \grave{m} * b_{F.S.} = 28.4 * 3.0 = 85.20\ kN.m$$

Strip at Short Direction.

Span $= 6.0\ m$

Width $= 7.0\ m$



$$M_o = \frac{(w_s * L_2)\,(L_1 - \frac{2}{3}D)^2}{8} = \frac{(14.20 * 7.0)\,(6.0 - \frac{2}{3} * 0.5)^2}{8}$$

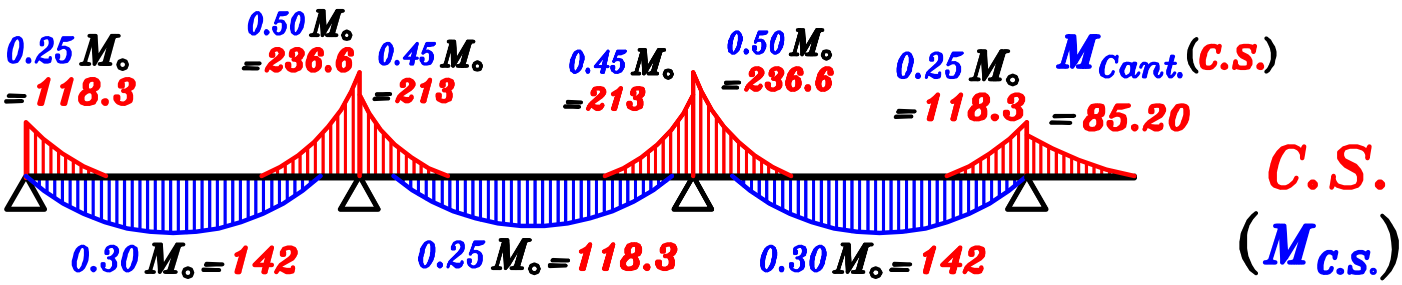
$$\boxed{M_o = 399.0\ kN.m}$$ Short Direction

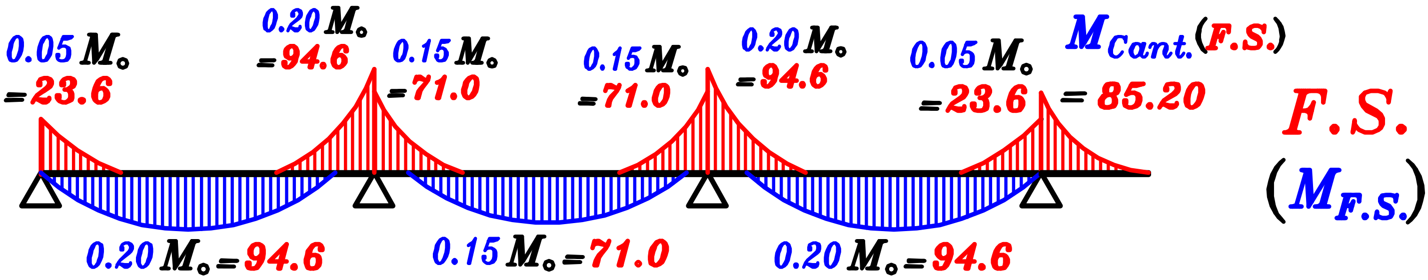
Distribute the B.M. (M_o) on C.S. & F.S.

Long Direction.

Column Strip width = Field Strip width $= \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$

$$\boxed{M_o = 473.3\ kN.m}$$ Long Direction



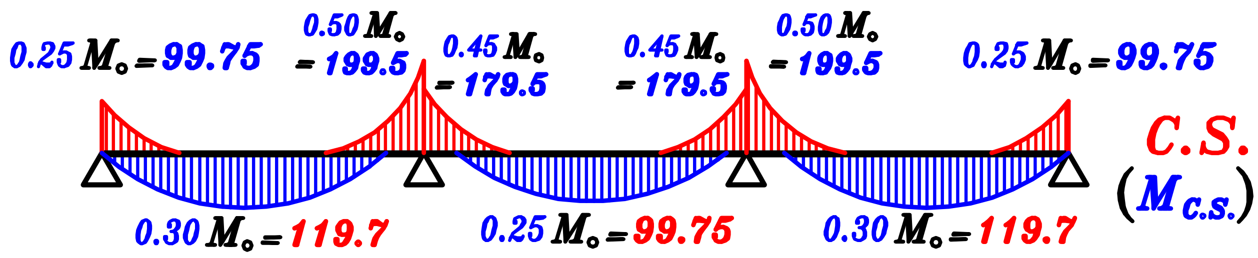


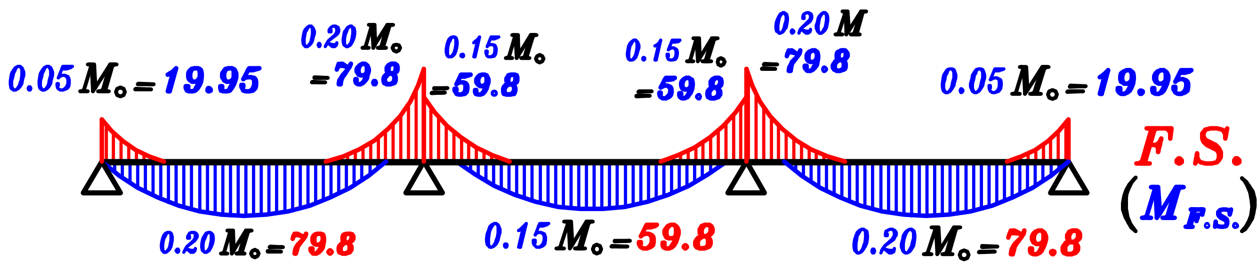
Short Direction.

$$\text{Column Strip width} = \frac{L_2}{2} = \frac{6.0}{2} = 3.0\ m$$

$$\text{Field Strip width} = L_1 - \frac{L_2}{2} = 7.0 - \frac{6.0}{2} = 4.0\ m$$

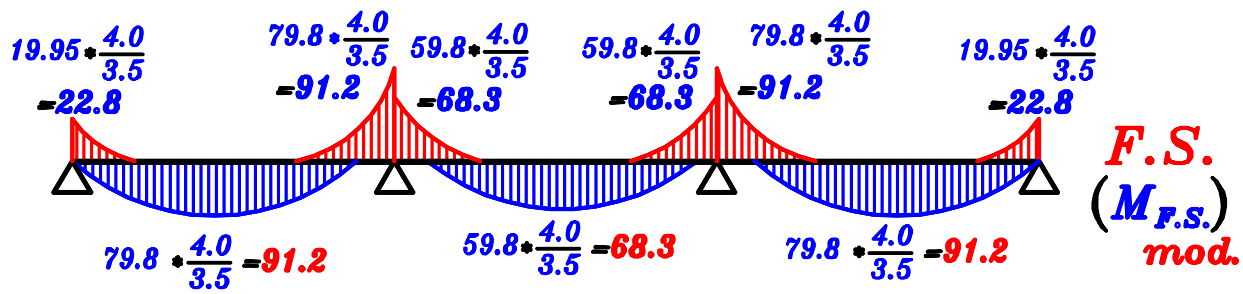
$$\boxed{M_o = 399.0\ kN.m}$$ **Short Direction**



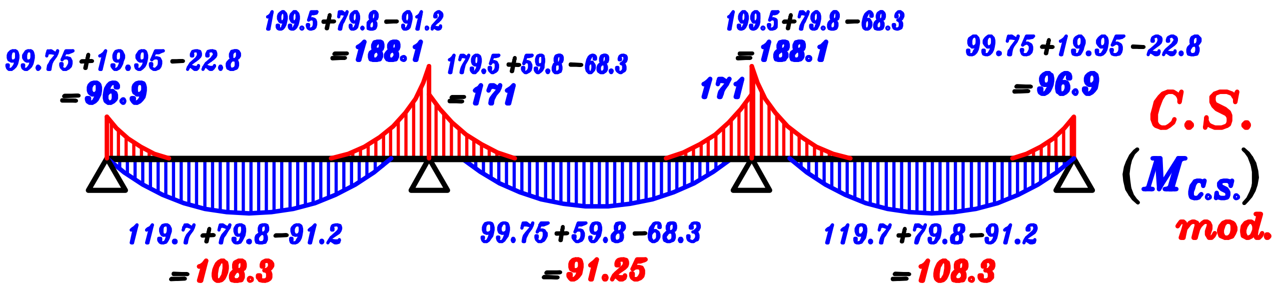


$$\text{Modification Factor} = \frac{L_1 - L_2\backslash 2}{L_1\backslash 2} = \frac{4.0}{3.5}$$

$$\boxed{(M_{F.S.})_{mod.} = (M_{F.S.}) * \text{Modification Factor} = (M_{F.S.}) * \frac{4.0}{3.5}}$$

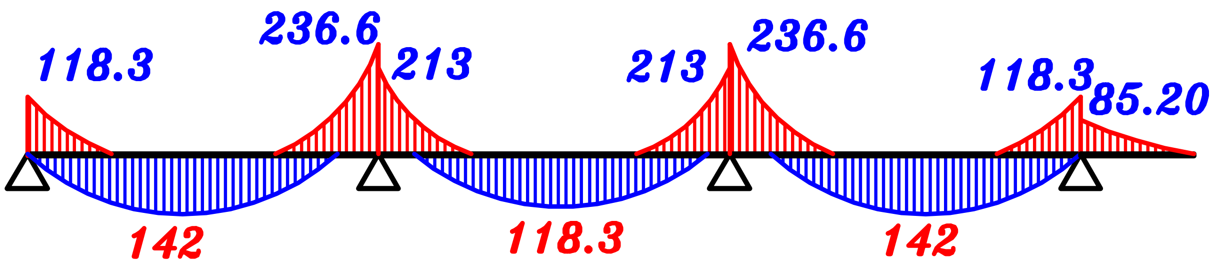


$$\boxed{(M_{C.S.})_{mod.} = (M_{C.S.}) + (M_{F.S.}) - (M_{F.S.})_{mod.}}$$

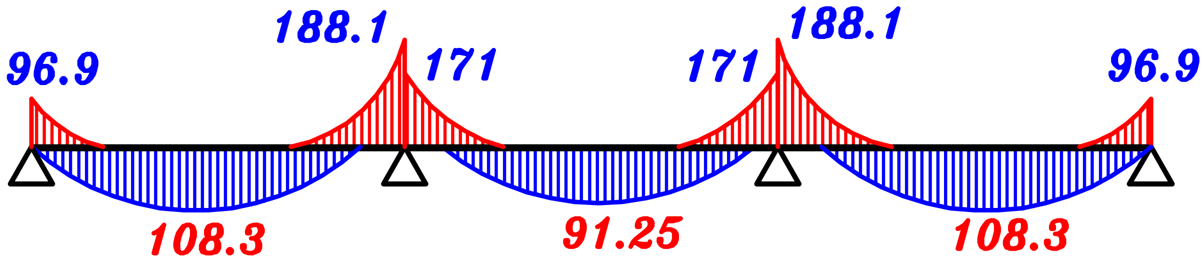


The moment of Column Strips.

Long Direction.



Short Direction.



Design of Columns.

$$w_s = 1.4\,(t_s\,\gamma_c + F.C. + Walls) + 1.6\,(L.L.)$$

$$w_s = 1.4\,(0.22 * 25 + 1.0 + 1.5) + 1.6\,(2.0) = 14.20\ kN\backslash m^2$$

$$g_s = 0.9\,(t_s\,\gamma_c + F.C. + Walls)$$

$$g_s = 0.9\,(0.22 * 25 + 1.0 + 1.5) = 7.20\ kN\backslash m^2$$

$$\boxed{w_s = 14.20\ kN\backslash m^2} \qquad \boxed{g_s = 7.20\ kN\backslash m^2}$$

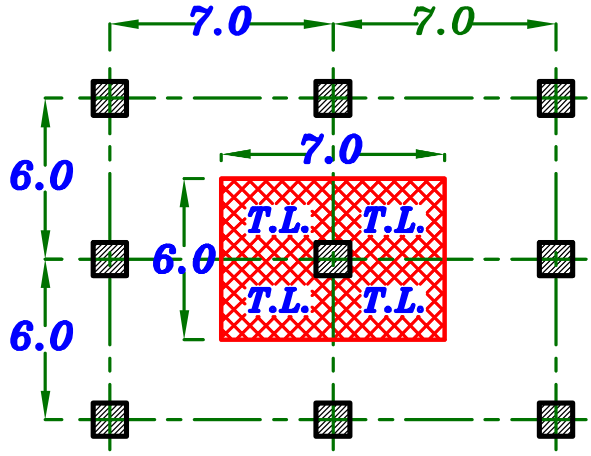
$$M_{Cant.(T.L.)} = \frac{w_s * L_c^2}{2} * b_{c.s.} = \frac{14.20 * (2.0)^2}{2} * 3.0 = 85.20\ kN.m$$

$$M_{Cant.(D.L.)} = \frac{g_s * L_c^2}{2} * b_{c.s.} = \frac{7.20 * (2.0)^2}{2} * 3.0 = 43.20\ kN.m$$

No. of Floors = 6 Floors

Interior Column. C_1 (For Ground Floor)

Total Load.



$$P\backslash Floor = w_s * (L_1 * L_2) * 1.1$$

$$P\backslash Floor = 14.20 * (7.0 * 6.0) * 1.1$$

$$= 656.0 \ kN$$

عدد الادوار

$$P_{(total)} = 656.0 * 6.0 = 3936.2 \ kN$$

$$M_{ext.} = Zero$$

لتحديد أبعاد العمود المبدئيه نفرض أن العمود عليه *axial load* فقط

$$P_{U.L.} = 0.35 A_c F_{cu} + 0.67 A_s F_y \xrightarrow{Take} \mu = \frac{A_s}{A_c} = 1.0\ \%$$

$$\therefore 3936.2 * 10^3 = 0.35 (A_c)(30) + 0.67 \left(\frac{A_c}{100}\right)(360)$$

$$\longrightarrow A_c = 304848 \ mm^2 \longrightarrow b = \sqrt{A_c} = \sqrt{304848} = 552.1 \ mm$$

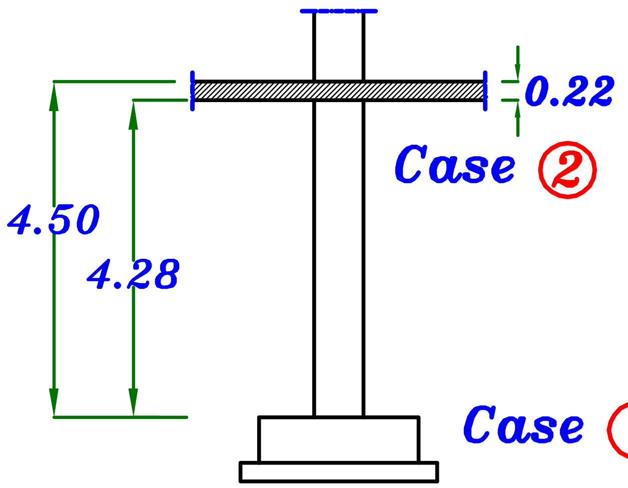
Take $\boxed{b = 600 \ mm}$

يجب أن لا تقل b عن 500 mm
حتى تكون البلاطه *Safe Punching*.

Check Buckling.

In plane & out of plane.

العمود متماثل فى الاتجاهين



Upper Case ②
Lower Case ①
$k = 1.3$

$$H_o = 4.28 \ m$$

$$\lambda_b = \frac{1.3 * 4.28}{0.60}$$

$$= 9.27 < 10$$

$\lambda_b < 10$ **Short Column.**

$$P_{U.L.} = 0.35\, A_c\, F_{cu} + 0.67\, A_s\, F_y$$

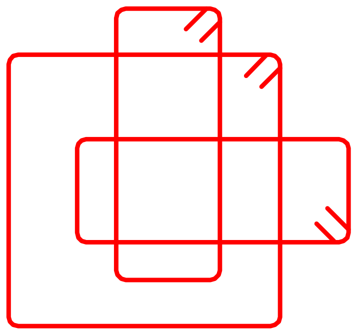
$$3936.2 * 10^3 = 0.35\, (600 * 600)\,(30) + 0.67\, A_s\,(360)$$

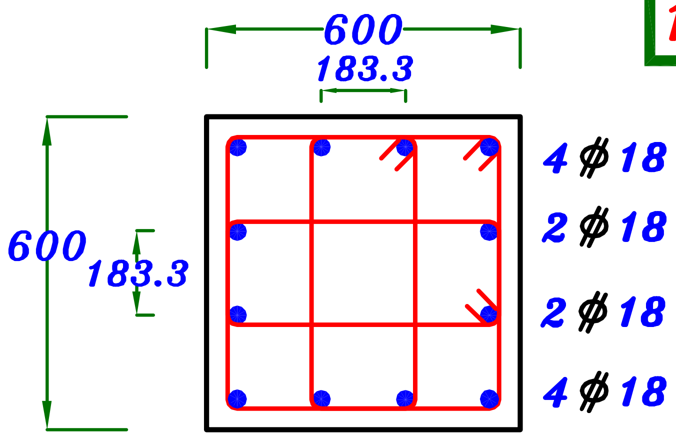
$$A_s = 647.6 \text{ mm}^2$$

Check $A_{s_{min}} = \frac{0.8}{100} * (600 * 600) = 2880 \text{ mm}^2$

$$A_s < A_{s_{min}} \xrightarrow{\text{Take}} A_s = A_{s_{min}} = 2880 \text{ mm}^2$$

$\boxed{12\ \phi\ 18}$



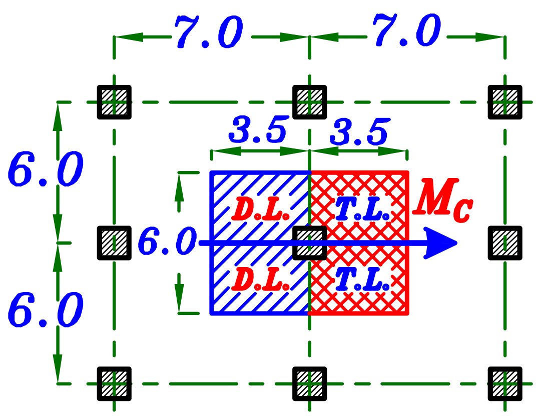


Interior Column. C_1 (For Last Floor)

$$P\backslash Floor = \left[w_s * \left(\frac{L_1}{2} * L_2\right) + g_s * \left(\frac{L_1}{2} * L_2\right)\right] * 1.1$$

$$= \left[14.20 * (3.5 * 6.0) + 7.20 * (3.5 * 6.0)\right] * 1.1$$

$$= 494.34 \text{ kN}$$



عدد الادوار التى يحملها عمود الدور الاخير

$$P\,(total) = 494.34 * 1.0 = 494.34 \text{ kN}$$

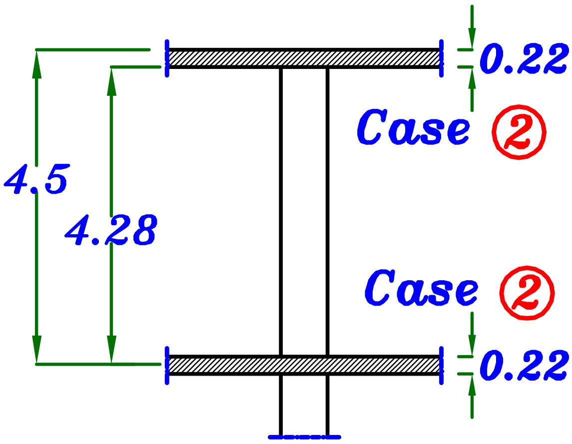
$$M_C = 50\,\%\, M_{C.S.} = 0.5 * (0.50\, M_o) = 0.5 * 236.6 = 118.3 \text{ kN.m}$$

$\boxed{M_{ext} = M_C = 118.3 \text{ kN.m}}$ يؤخذ العزم كما هو لانه أخر دور

Take the column $(500 * 500)$

> يجب أن لا تقل b عن 500 mm
> حتى تكون البلاطه Safe Punching.

Check Buckling. In plane & out of plane.



العمود متماثل فى الاتجاهين

Upper Case ②
Lower Case ②
$\Big\}$ $k = 1.5$

$H_o = 4.28$ m

$$\lambda_b = \frac{1.5 * 4.28}{0.50} = 12.84 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{12.84^2 * 0.5}{2000} = 0.041 \text{ m}$$

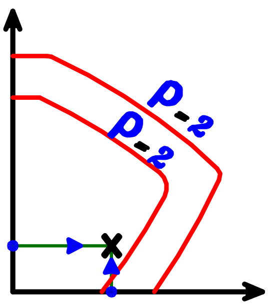
$$M_{add.} = P * \delta = 494.34 * 0.041 = 20.26 \text{ kN.m}$$

$$M_{des.} = M_{ext.} + M_{add.} = 118.3 + 20.26 = 138.56 \text{ kN.m}$$

$$e = \frac{M}{P} = \frac{138.56}{494.34} = 0.28 \text{ m} \qquad \therefore \frac{e}{t} = \frac{0.28}{0.50} \simeq 0.50 \xrightarrow{use} I.D.$$

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page 4–24}$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{494.34 * 10^3}{30 * 500 * 500} = 0.066 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{138.56 * 10^6}{30 * 500 * 500^2} = 0.037 \end{aligned}\right\} \rho = 1.0$$



$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 3.0 * 10^{-3}$$

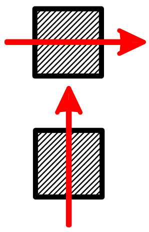
$$A_s = A_s` = \mu * b * t = 3.0 * 10^{-3} * 500 * 500 = 750 \text{ mm}^2$$

$$A_{s\,Total} = A_s + A_s` = 2 * 750 = 1500 \text{ mm}^2$$

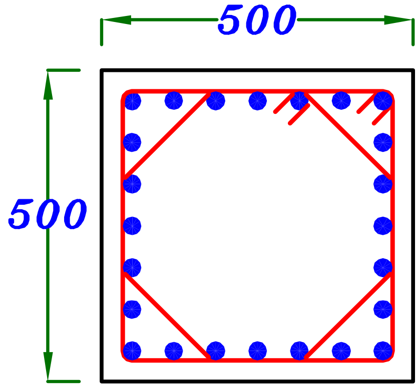
$$A_{s_{min}} = \frac{0.25 + 0.052\, \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\,(12.84)}{100} * 500 * 500 = 2294.2 \text{ mm}^2 > A_{s\,total}$$

$$\text{Take } A_s = A_s` = \frac{A_{s_{min}}}{2} = 1147.1 \text{ mm}^2 \qquad \boxed{6 \phi 16}$$



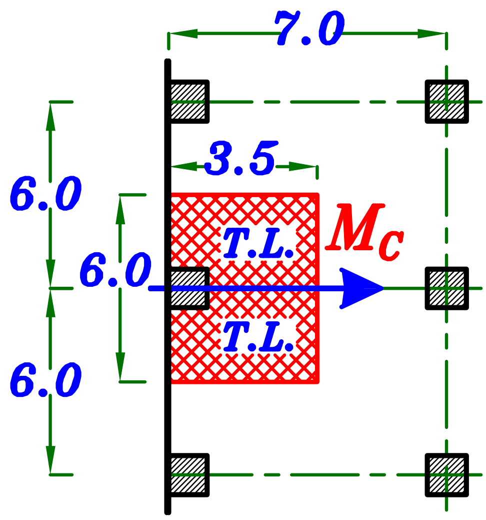
يتم وضع التسليح فى الاتجاهين بنفس القيمه
لانه قد تم التصميم على اتجاه واحد فقط
و لم يتم عمل حالات تحميل للاتجاه الاخر .



$24\phi 16$

عدد الاسياخ ٦ اسياخ فى كل جنب
المجموع الكلى للاسياخ ٢٤ سيخ
نضع ٤ اسياخ فى الاركان و الباقى يوزع على الاربع جناب .

Edge Column. C_2 عمود طرفى (For Ground Floor)



$$P\backslash Floor = w_s * \left(\frac{L_1}{2} * L_2\right) * 1.1$$

$$P\backslash Floor = 14.20 * (3.5 * 6.0) * 1.1$$

$$= 328.0 \ kN$$

عدد الادوار

$$P\ (total) = 328.0 * 6.0 = 1968 \ kN$$

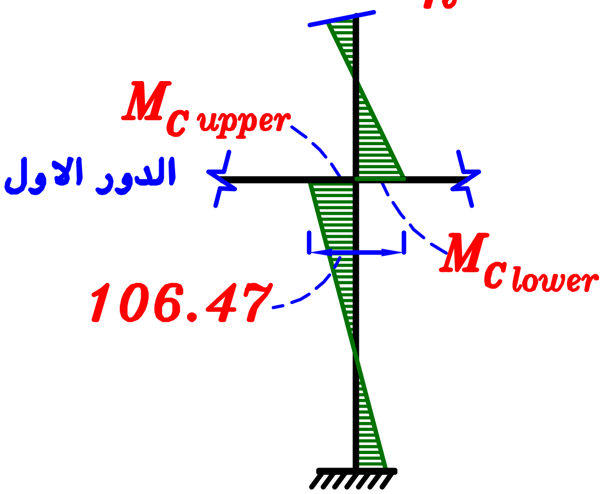
$$M_C = 90\ \%\ M_{c.s.} = 0.9 * (0.25\ M_o)$$ Without marginal beam

$$M_C = 0.9 * (118.3) = 106.47 \ kN.m$$

و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

لان نسبه $\frac{I}{h}$ متساويه للعمود السفلى و العلوى

فيتم توزيع العزم على العمودين بالتساوى



$$\boxed{M_{ext} = \frac{M_C}{2} = 53.2 \ kN.m}$$

لتحديد أبعاد العمود المبدئيه نفرض أن العمود عليه *axial load* فقط

$$P_{U.L.} = 0.35\,A_c\,F_{cu} + 0.67\,A_s\,F_y \xrightarrow{Take} \mu = \frac{A_s}{A_c} = 1.0\ \%$$

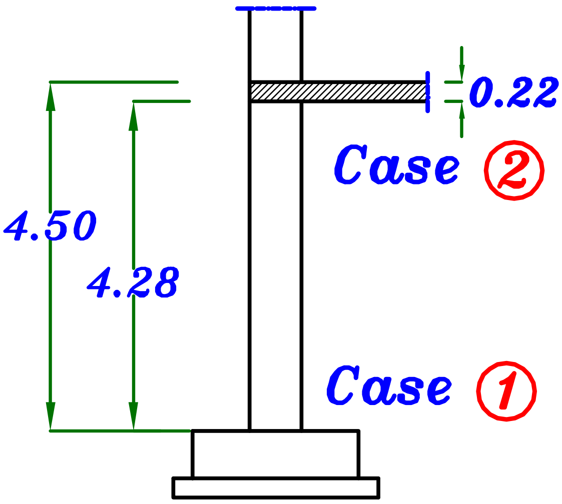
$$\therefore 1968 * 10^3 = 0.35\,(A_c)(30) + 0.67\left(\frac{A_c}{100}\right)(360)$$

$$\rightarrow A_c = 152416\ mm^2 \rightarrow b = \sqrt{A_c} = \sqrt{152416} = 390\ mm$$

Take $\boxed{b = 500\ mm}$

يجب أن لا تقل b عن $500\ mm$
حتى تكون البلاطه *Safe Punching*.

Check Buckling.



In plane & out of plane.

العمود متماثل فى الاتجاهين

Upper Case 2
Lower Case 1 $\Big\}$ $k = 1.3$

$$H_o = 4.28\ m$$

$$\lambda_b = \frac{1.3 * 4.28}{0.50} = 11.13 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{11.13^2 * 0.5}{2000} = 0.031\ m$$

$$M_{add.} = P * \delta = 1968 * 0.031 = 61.0\ kN.m$$

$$M_{des.} = M_{ext.} + M_{add.} = 53.2 + 61.0 = 114.2\ kN.m$$

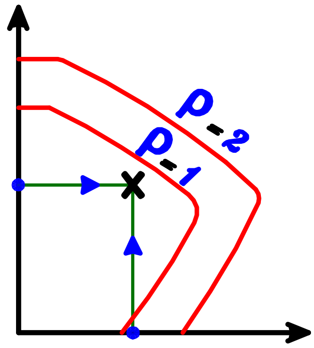
$$e = \frac{M}{P} = \frac{114.2}{1968} = 0.058\ m \qquad \therefore \frac{e}{t} = \frac{0.058}{0.50} = 0.116 \xrightarrow{use} I.D.$$

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page 4-24}$$

$$\frac{P_U}{F_{cu}\, b\, t} = \frac{1968 * 10^3}{30*500*500} = 0.26$$

$$\frac{M_U}{F_{cu}\, b\, t^2} = \frac{114.2 * 10^6}{30*500*500^2} = 0.03$$

$$\rho = 1.0$$



$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 3.0 * 10^{-3}$$

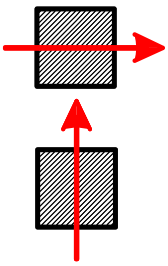
$$A_s = A_{s`} = \mu * b * t = 3.0 * 10^{-3} * 500 * 500 = 750\ mm^2$$

$$A_{s\,Total} = A_s + A_{s`} = 2 * 750 = 1500\ mm^2$$

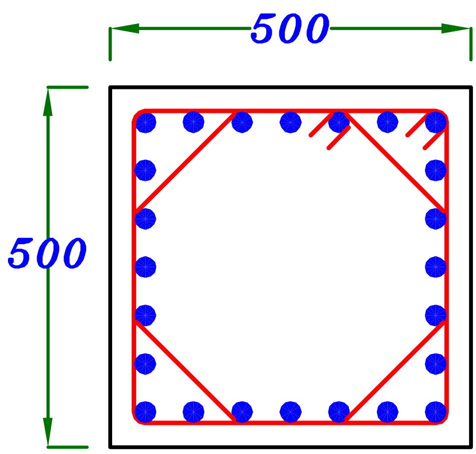
$$A_{s_{min}} = \frac{0.25 + 0.052\, \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\,(11.13)}{100} * 500 * 500 = 2071.9\ mm^2 > A_{s\,total}$$

Take $A_s = A_{s`} = \frac{A_{s_{min}}}{2} = 1035.9\ mm^2$ $\boxed{6 \phi 16}$



يتم وضع التسليح فى الاتجاهين بنفس القيمه
لانه قد تم التصميم على اتجاه واحد فقط
و لم يتم عمل حالات تحميل للاتجاه الاخر .



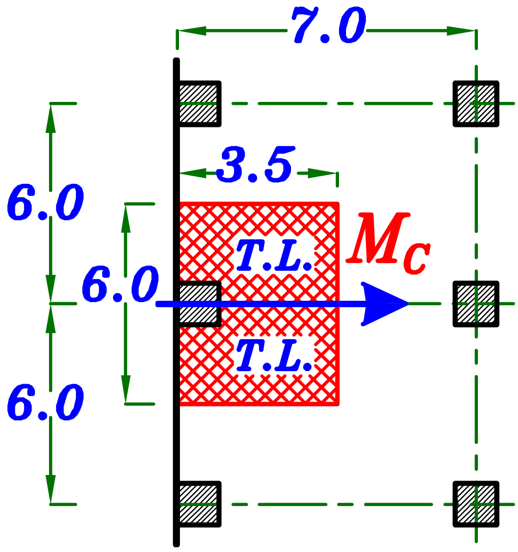
عدد الاسياخ ٦ اسياخ فى كل جنب

المجموع الكلى للاسياخ ٢٤ سيخ

نضع ٤ اسياخ فى الاركان و الباقى يوزع على الاربع جناب .

$24 \phi 16$

Edge Column. C_2 عمود طرفى (For Last Floor)



$$P\backslash Floor = w_s * \left(\frac{L_1}{2} * L_2\right) * 1.1$$

$$P\backslash Floor = 14.20 * (3.5 * 6.0) * 1.1$$

$$= 328.0 \ kN$$

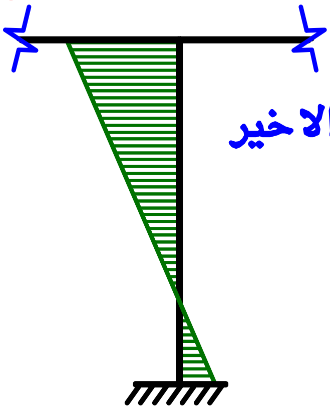
عدد الادوار التى يحملها عمود الدور الاخير

$$P\,(total) = 328.0 * 1.0 = 328.0 \ kN$$

$$M_C = 90\,\%\, M_{c.s.} = 0.9 * (0.25\, M_o)$$ Without marginal beam

$$M_C = 0.9 * (118.3) = 106.47 \ kN.m$$

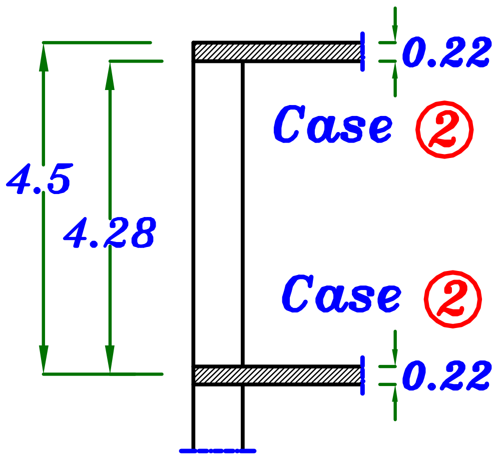
$$\boxed{M_{ext} = M_C = 106.47 \ kN.m}$$ يؤخذ العزم كما هو لانه أخر دور



يجب أن لا تقل b عن $500\ mm$
حتى تكون البلاطه Safe Punching.

Take $\boxed{b = 500\ mm}$

Check Buckling.



العمود متماثل فى الاتجاهين

Upper Case ②
Lower Case ②
$k = 1.5$

$$H_o = 4.28 \ m$$

$$\lambda_b = \frac{1.5 * 4.28}{0.50} = 12.84 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{12.84^2 * 0.5}{2000} = 0.041 \ m$$

$$M_{add.} = P * \delta = 328.0 * 0.041 = 13.45 \ kN.m$$

$$M_{des.} = M_{ext.} + M_{add.} = 106.47 + 13.45 = 119.9 \ kN.m$$

$$e = \frac{M}{P} = \frac{119.9}{328.0} = 0.365 \, m \qquad \therefore \frac{e}{t} = \frac{0.365}{0.50} = 0.73 \xrightarrow{use} e_s$$

$$e_s = e + \frac{t}{2} - c = 0.365 + \frac{0.50}{2} - 0.05 = 0.565 \ m$$

$$M_s = P * e_s = 328.0 * 0.565 = 185.32 \ kN.m$$

$$\therefore d = C_1 \sqrt{\frac{M_s}{F_{cu} b}} \qquad \therefore 450 = C_1 \sqrt{\frac{185.32 * 10^6}{30 * 500}} \rightarrow C_1 = 4.04 \rightarrow J = 0.804$$

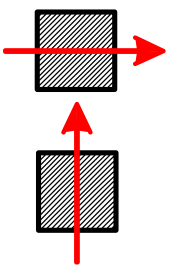
$$\therefore A_s = \frac{M_s}{J \, F_y \, d} - \frac{P_{U.L.}}{(F_y \backslash \gamma_s)}$$

$$= \frac{185.32 * 10^6}{0.804 * 360 * 450} - \frac{328.0 * 10^3}{(360 \backslash 1.15)} = 375.0 \ mm^2$$

$$A_{s_{min}} = \frac{0.25 + 0.052 \, \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052 \, (12.84)}{100} * 500 * 500 = 2294.2 \ mm^2 > A_{s_{total}}$$

$$\boxed{6 \, \phi \, 22}$$

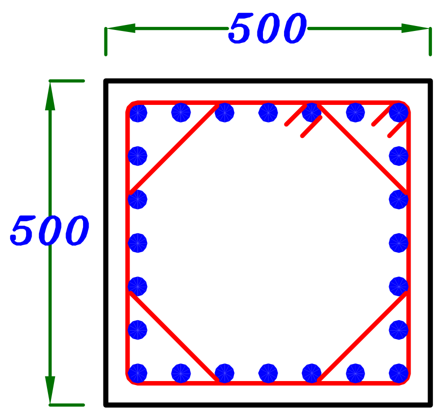


يتم وضع التسليح فى الاتجاهين بنفس القيمه
لانه قد تم التصميم على اتجاه واحد فقط
و لم يتم عمل حالات تحميل للاتجاه الاخر .

عدد الاسياخ ٦ اسياخ فى كل جنب

المجموع الكلى للاسياخ ٢٤ سيخ

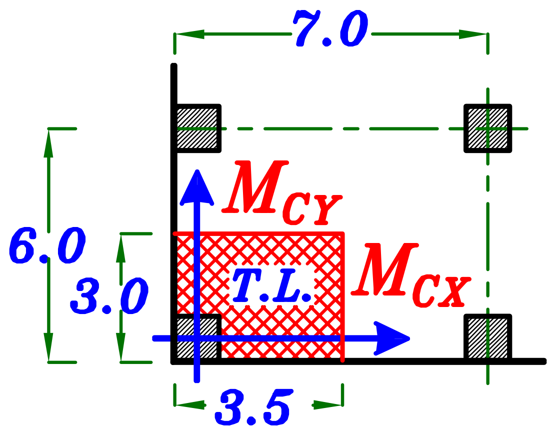
نضع ٤ اسياخ فى الاركان و الباقى يوزع على الاربع جناب .



$24 \phi 22$

Corner Column. عمود ركنى (For Ground Floor)

$$P\backslash Floor = w_s * \left(\frac{L_1}{2} * \frac{L_2}{2}\right) * 1.1$$



$$P\backslash Floor = 14.20 * (3.5 * 3.0) * 1.1$$

$$= 164.0 \ kN$$

عدد الادوار

$$P_{(total)} = 164.0 * 6.0 = 984.0 \ kN$$

$$M_{CX} = 0.5 * 90\% \ M_{c.s.} = 0.5 * 0.9 * (0.25 \ M_o) \quad \text{For H.L. Strip}$$

$$M_{CX} = 0.5 * 0.9 * (118.3) = 53.2 \ kN.m$$

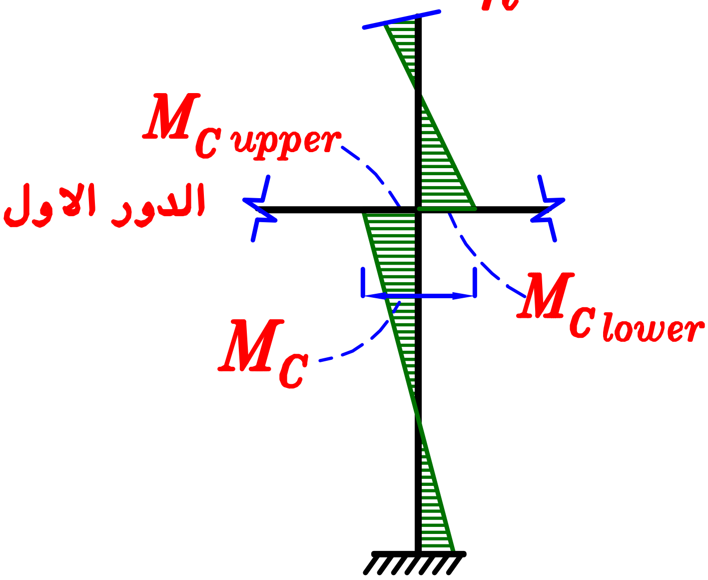
$$M_{CY} = 0.5 * 90\% \ M_{c.s.} = 0.5 * 0.9 * (0.25 \ M_o) \quad \text{For V.L. Strip}$$

$$M_{CX} = 0.5 * 0.9 * (93.1) = 41.89 \ kN.m$$

و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

لان نسبه $\frac{I}{h}$ متساويه للعمود السفلى و العلوى

فيتم توزيع العزم على العمودين بالتساوى



$$\boxed{M_{ext\,X} = \frac{M_{CX}}{2} = 26.60 \ kN.m}$$

$$\boxed{M_{ext\,Y} = \frac{M_{CY}}{2} = 20.94 \ kN.m}$$

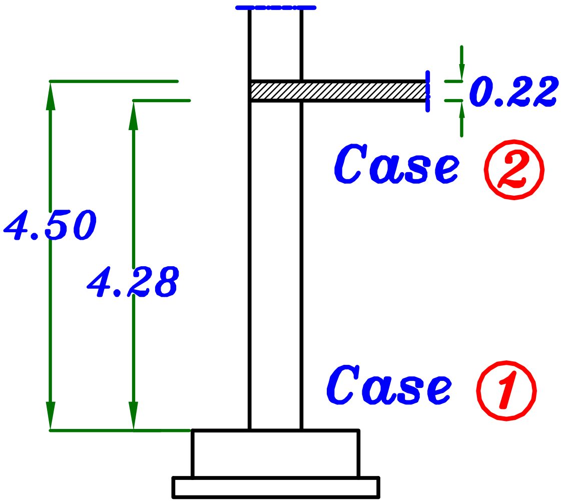
Take $\boxed{b = 500 \ mm}$

يجب أن لا تقل b عن 500 mm
حتى تكون البلاطه Safe Punching .

Check Buckling.

In plane & out of plane.

العمود متماثل فى الاتجاهين



Upper Case ②
Lower Case ①
$\Big\}\ k = 1.3$

$H_o = 4.28\ m$

$$\lambda_b = \frac{1.3 * 4.28}{0.50} = 11.13 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{11.13^2 * 0.5}{2000} = 0.031\ m$$

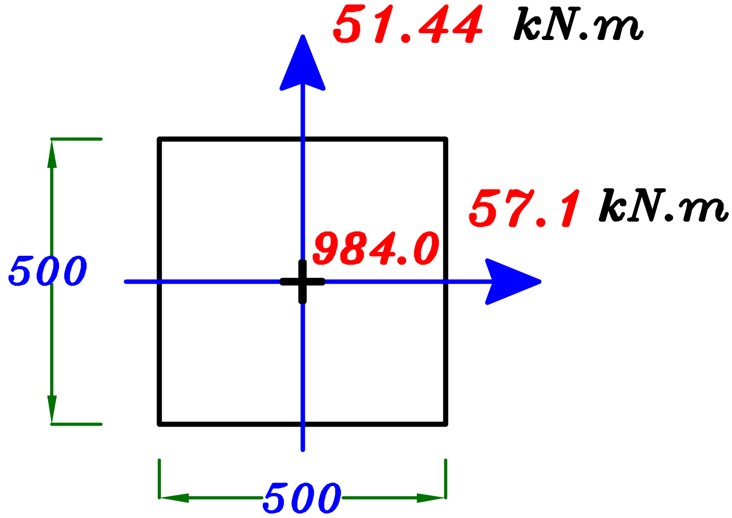
$$M_{add.} = P * \delta = 984.0 * 0.031 = 30.5\ kN.m$$

$$M_{X\,des.} = M_{X\,ext.} + M_{add.} = 26.60 + 30.5 = 57.1\ kN.m$$

$$M_{Y\,des.} = M_{Y\,ext.} + M_{add.} = 20.94 + 30.5 = 51.44\ kN.m$$

$$\frac{M_X}{a} = \frac{57.1}{0.45} = 126.9$$

$$\frac{M_Y}{b} = \frac{51.44}{0.45} = 114.3$$

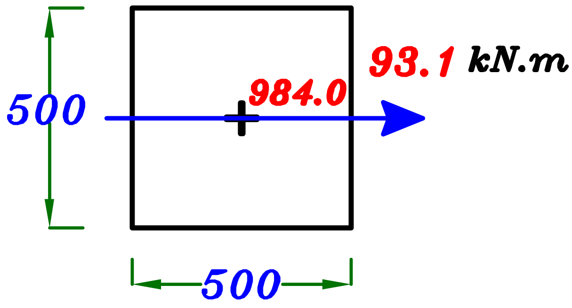


$$\frac{M_X}{a} > \frac{M_Y}{b} \longrightarrow$$ Neglect M_Y and design the Sec. on M_X

$$\boxed{M_{X`} = M_X + \beta \left(\frac{a}{b}\right) M_Y} \longrightarrow \text{take } \beta = 0.70$$

$$M_{x'} = 57.1 + 0.70\left(\frac{0.45}{0.45}\right) 51.44 = 93.10 \ kN.m$$

$$e = \frac{M}{P} = \frac{93.1}{984.0} = 0.094 \ m$$



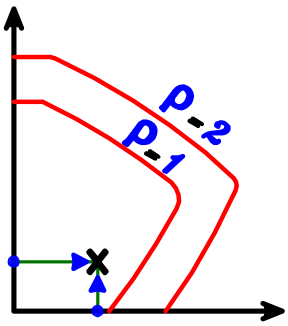
$$\frac{e}{t} = \frac{0.094}{0.50} = 0.19 < 0.5 \xrightarrow{use} I.D.$$

ملحوظه فى حاله Bi-Axial Moment يجب استخدام I.D.

$\therefore$ Use Interaction Diagram

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page 4-24}$$

$$\left.\begin{aligned} \frac{P_U}{F_{cu}\, b\, t} &= \frac{984.0 * 10^3}{30*500*500} = 0.131 \\ \frac{M_U}{F_{cu}\, b\, t^2} &= \frac{93.1 * 10^6}{30*500*500^2} = 0.025 \end{aligned}\right\} \rho = 1.0$$



$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 3.0 * 10^{-3}$$

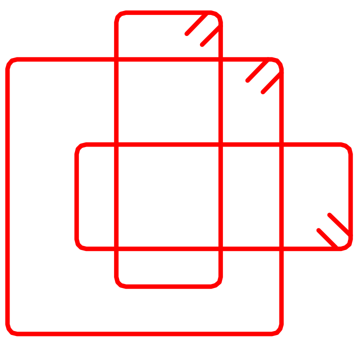
$$A_s = A_s' = \mu * b * t = 3.0 * 10^{-3} * 500 * 500 = 750 \ mm^2$$

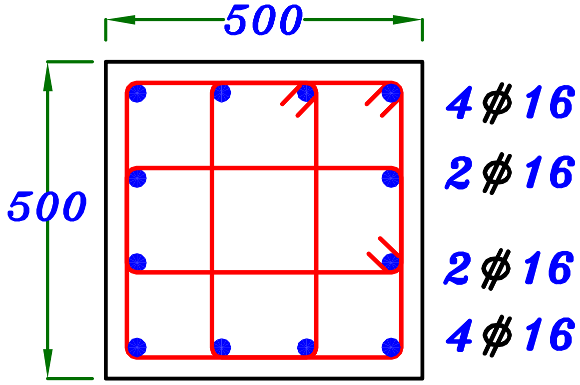
$$A_{s\,Total} = A_s + A_s' = 2 * 750 = 1500 \ mm^2$$

$$A_{s\,min} = \frac{0.25 + 0.052\,\lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\,(11.13)}{100} * 500 * 500 = 2072 \ mm^2 > A_{s\,total}$$

Take $A_{s\,total} = A_{s\,min} = 2072 \ mm^2$ 12 ϕ 16



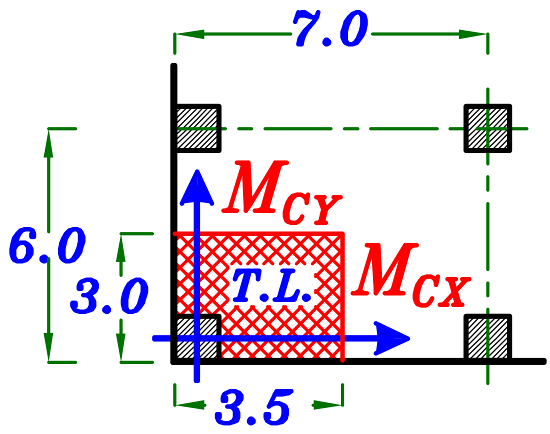


Corner Column. عمود ركنى (For Last Floor)

$$P\backslash Floor = w_S * \left(\frac{L_1}{2} * \frac{L_2}{2}\right) * 1.1$$

$$P\backslash Floor = 14.20 * (3.5 * 3.0) * 1.1$$

$$= 164.0 \ kN$$



عدد الادوار التى يحملها عمود الدور الاخير

$$P\ (total) = 164.0 * 1.0 = 164.0 \ kN$$

$$M_{CX} = 0.5 * 90\% \ M_{c.s.} = 0.5 * 0.9 * (0.25 \ M_o)$$ For H.L. Strip

$$M_{CX} = 0.5 * 0.9 * (118.3) = 53.2 \ kN.m$$

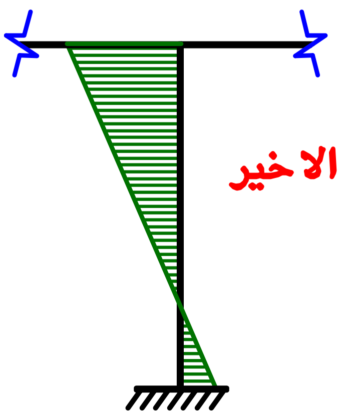
$$M_{CY} = 0.5 * 90\% \ M_{c.s.} = 0.5 * 0.9 * (0.25 \ M_o)$$ For V.L. Strip

$$M_{CX} = 0.5 * 0.9 * (93.1) = 41.89 \ kN.m$$

$$\boxed{M_{ext\,X} = M_{CX} = 53.20 \ kN.m}$$

يؤخذ العزم كما هو لانه أخر دور

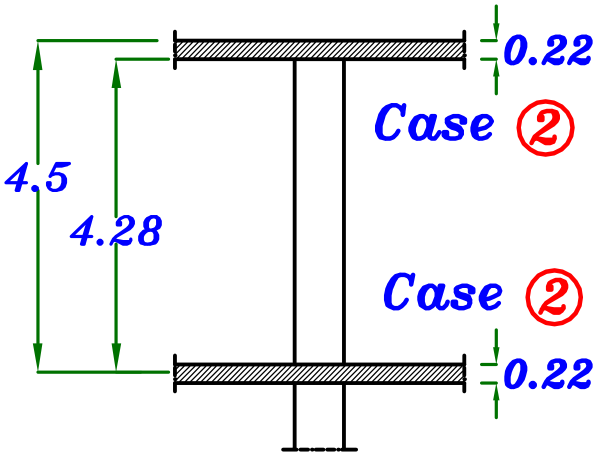
$$\boxed{M_{ext\,Y} = M_{CY} = 41.89 \ kN.m}$$



يجب أن لا تقل b عن 500 mm
حتى تكون البلاطه Safe Punching.

Take $\boxed{b = 500 \ mm}$

Check Buckling. In plane & out of plane.



العمود متماثل فى الاتجاهين

Upper Case ②
Lower Case ②
$\Big\}\ k = 1.5$

$H_o = 4.28\ m$

$$\lambda_b = \frac{1.5 * 4.28}{0.50} = 12.84 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{12.84^2 * 0.5}{2000} = 0.041\ m$$

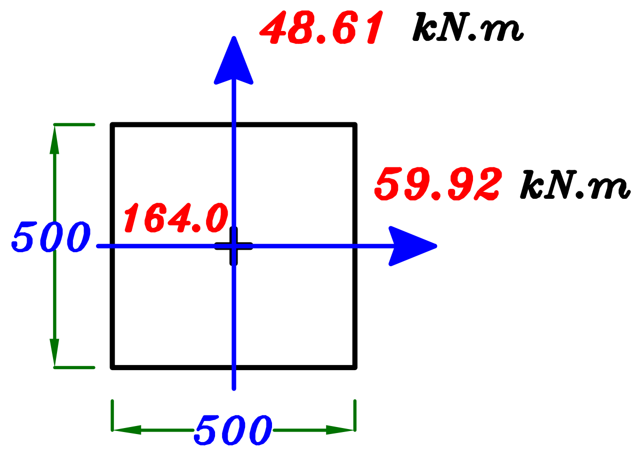
$$M_{add.} = P * \delta = 164.0 * 0.041 = 6.72\ kN.m$$

$$M_{X\,des.} = M_{X\,ext.} + M_{add.} = 53.20 + 6.72 = 59.92\ kN.m$$

$$M_{Y\,des.} = M_{Y\,ext.} + M_{add.} = 41.89 + 6.72 = 48.61\ kN.m$$

$$\frac{M_X}{a} = \frac{59.92}{0.45} = 133.15$$

$$\frac{M_Y}{b} = \frac{48.61}{0.45} = 108.02$$



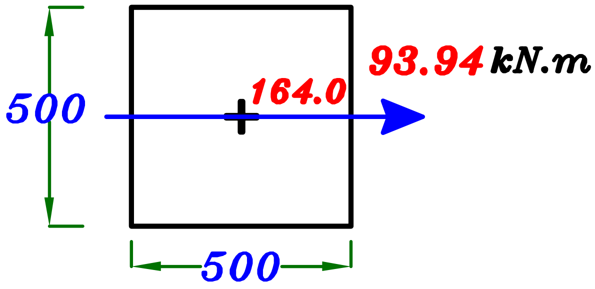
$$\frac{M_X}{a} > \frac{M_Y}{b} \longrightarrow$$ Neglect M_Y and design the Sec. on M_X

$$\boxed{M_{X`} = M_X + \beta \left(\frac{a}{b}\right) M_Y} \longrightarrow \text{take } \beta = 0.70$$

$$M_{x`} = 59.92 + 0.70\left(\frac{0.45}{0.45}\right) 48.61 = 93.94 \ kN.m$$

$$e = \frac{M}{P} = \frac{93.94}{164.0} = 0.572 \ m$$

$$\frac{e}{t} = \frac{0.572}{0.50} = 1.14 > 0.5$$



I.D. ملحوظه فى حاله Bi-Axial Moment يجب استخدام

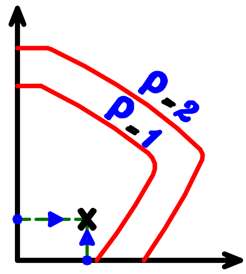
$\therefore$ *Use Interaction Diagram*

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page } 4\text{–}24$$

$$\frac{P_U}{F_{cu}\, b\, t} = \frac{164.0 * 10^3}{30 * 500 * 500} = 0.021$$

$$\frac{M_U}{F_{cu}\, b\, t^2} = \frac{93.94 * 10^6}{30 * 500 * 500^2} = 0.025$$

$$\rho = 1.0$$



$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 3.0 * 10^{-3}$$

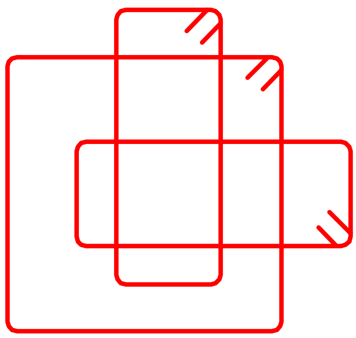
$$A_s = A_{s`} = \mu * b * t = 3.0 * 10^{-3} * 500 * 500 = 750 \ mm^2$$

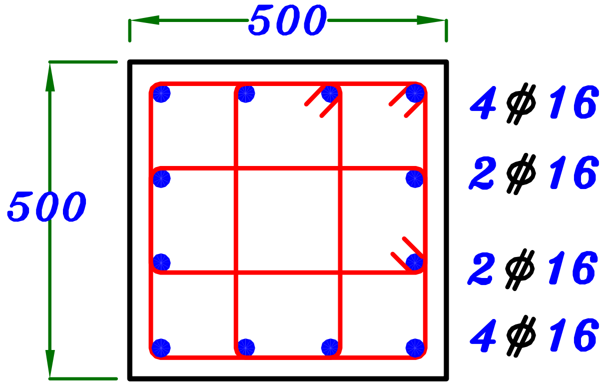
$$A_{s\,Total} = A_s + A_{s`} = 2 * 750 = 1500 \ mm^2$$

$$A_{s\,min} = \frac{0.25 + 0.052\,\lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\,(12.84)}{100} * 500 * 500 = 2294 \ mm^2 > A_{s\,total}$$

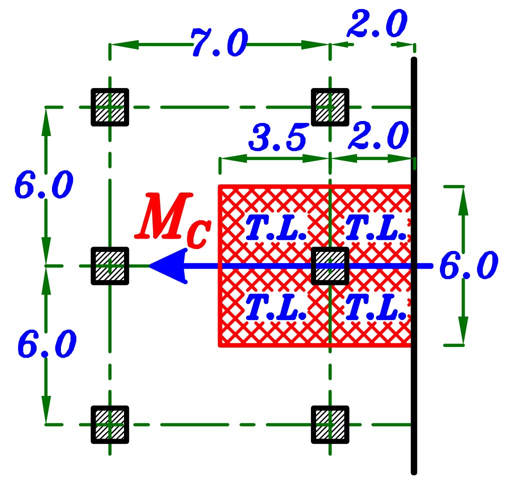
$$\text{Take } A_{s\,total} = A_{s\,min} = 2294 \ mm^2 \quad 12\phi16$$





Column at cantilever. عمود عند الكابولى

(For Ground Floor)



$$P\backslash Floor = w_s * \left[\left(\frac{L_1}{2} + L_c\right) * L_2\right] * 1.1$$

$$P\backslash Floor = 14.20 * \left[(3.5 + 2.0) * 6.0\right] * 1.1$$

$$= 515.46 \ kN$$

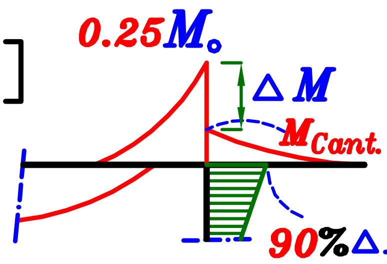
عدد الادوار

$$P_{(total)} = 515.46 * 6.0 = 3092.7 \ kN$$

$$M_{Cant.(T.L.)} = \frac{w_s * L_c^2}{2} * b_{c.s.} = \frac{14.20 * 2.0^2}{2} * 3.0 = 85.2 \ kN.m$$

$$M_C = 90\% (\Delta M) = 0.9 * \left[0.25 M_o - M_{Cant.(T.L.)}\right]$$

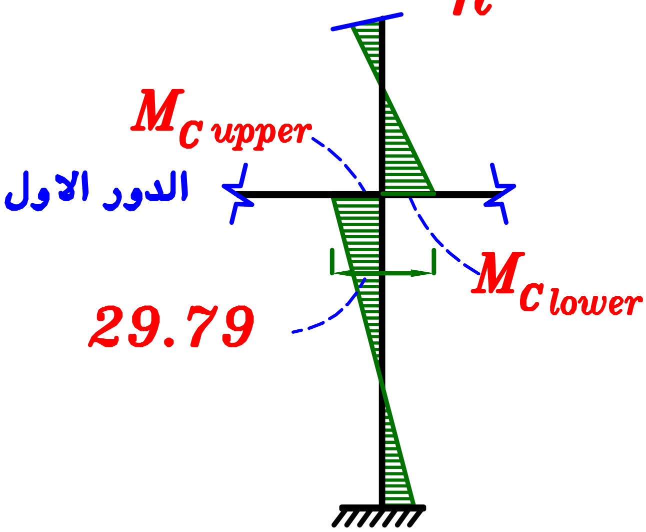
$$= 0.9 * \left[118.3 - 85.2\right] = 29.79 \ kN.m$$



و يوزع العزم M_C على العمودين السفلى و العلوى بنسبه $\frac{I}{h}$

لان نسبه $\frac{I}{h}$ متساويه للعمود السفلى و العلوى

فيتم توزيع العزم على العمودين بالتساوى



$$\boxed{M_{ext} = \frac{M_C}{2} = 14.9 \ kN.m}$$

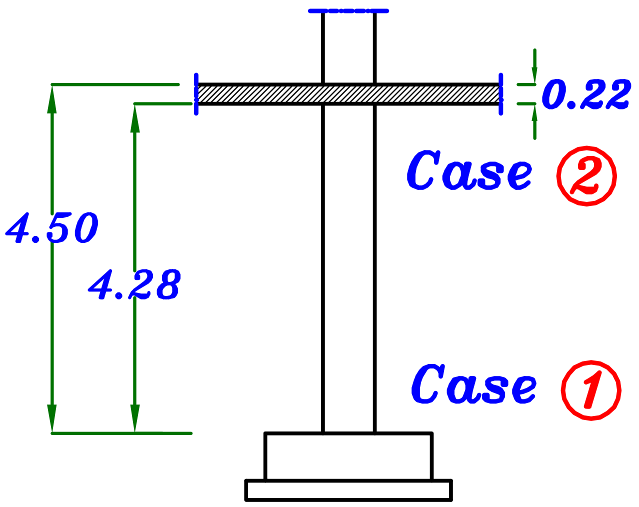
Take the column

(500 * 500)

يجب أن لا تقل b عن 500 mm

حتى تكون البلاطه Safe Punching.

Check Buckling. In plane & out of plane.



العمود متماثل فى الاتجاهين

Upper Case ② } $k = 1.3$
Lower Case ① }

$H_o = 4.28\ m$

$$\lambda_b = \frac{1.3 * 4.28}{0.50} = 11.13 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{11.13^2 * 0.5}{2000} = 0.031\ m$$

$$M_{add.} = P * \delta = 3092.7 * 0.031 = 95.87\ kN.m$$

$$M_{des.} = M_{ext.} + M_{add.} = 14.9 + 95.87 = 110.77\ kN.m$$

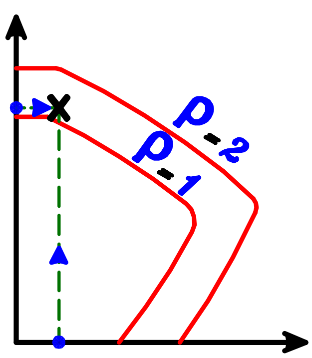
$$e = \frac{M}{P} = \frac{110.77}{3092.7} = 0.036\ m \quad \therefore \frac{e}{t} = \frac{0.036}{0.50} = 0.072 \xrightarrow{use} I.D.$$

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use} \text{ECCS Design Aids Page 4–24}$$

$$\frac{P_U}{F_{cu}\, b\, t} = \frac{3092.7 * 10^3}{30 * 500 * 500} = 0.412$$

$$\frac{M_U}{F_{cu}\, b\, t^2} = \frac{110.77 * 10^6}{30 * 500 * 500^2} = 0.03$$

$\rho = 1.1$



$$\mu = \rho * F_{cu} * 10^{-4} = 1.1 * 30 * 10^{-4} = 3.3 * 10^{-3}$$

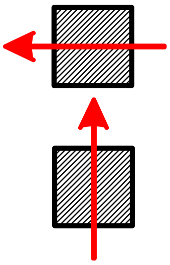
$$A_s = A_{s'} = \mu * b * t = 3.3 * 10^{-3} * 500 * 500 = 825\ mm^2$$

$$A_{s\,Total} = A_s + A_{s'} = 2 * 825 = 1650\ mm^2$$

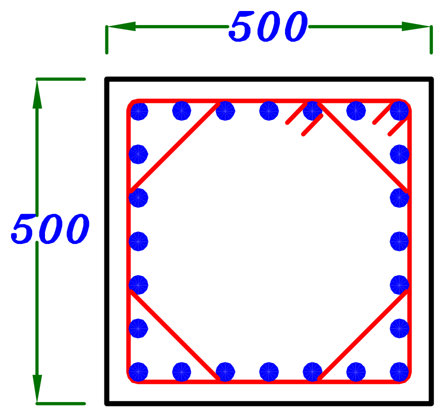
$$A_{s_{min}} = \frac{0.25 + 0.052\,\lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052\,(11.13)}{100} * 500 * 500 = 2071.9\ mm^2 > A_{s\,total}$$

Take $A_s = A_s' = \frac{A_{s_{min}}}{2} = 1035.9\ mm^2$ $\boxed{6\ \phi\ 16}$



يتم وضع التسليح فى الاتجاهين بنفس القيمه
لانه قد تم التصميم على اتجاه واحد فقط
و لم يتم عمل حالات تحميل للاتجاه الاخر.



عدد الاسياخ ٦ اسياخ فى كل جنب

المجموع الكلى للاسياخ ٢٤ سيخ

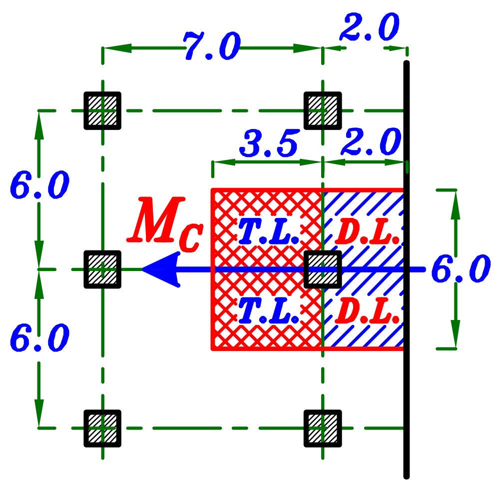
نضع ٤ اسياخ فى الاركان و الباقى يوزع على الاربع جناب .

$24\ \phi\ 16$

Column at cantilever. عمود عند الكابولى

(For Last Floor)

$$P\backslash Floor = \left[w_s\left(\frac{L_1}{2} * L_2\right) + g_s\,(L_c * L_2)\right] * 1.1$$



$P\backslash Floor =$

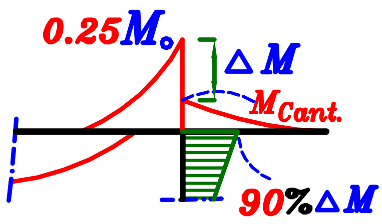
$$\left[14.20 * (3.5 * 6.0) + 7.20 * (2.0 * 6.0)\right] * 1.1$$

$$= 423.06\ kN$$

عدد الادوار التى يحملها عمود الدور الاخير

$$P\,(total) = 423.06 * 1.0 = 423.06\ kN$$

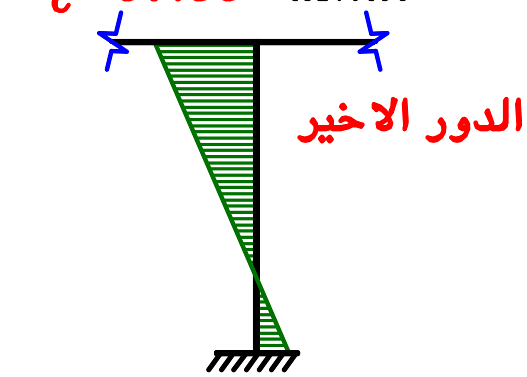
$$M_{Cant.}(D.L.) = \frac{g_s * L_c^2}{2} * b_{c.s.} = \frac{7.20 * 2.0^2}{2} * 3.0 = 43.2\ kN.m$$



$$M_C = 90\,\%\,(\Delta M) = 0.9 * \left[0.25\,M_o - M_{Cant.}(T.L.)\right]$$

$$= 0.9 * \left[118.3 - 43.2\right] = 67.59\ kN.m$$

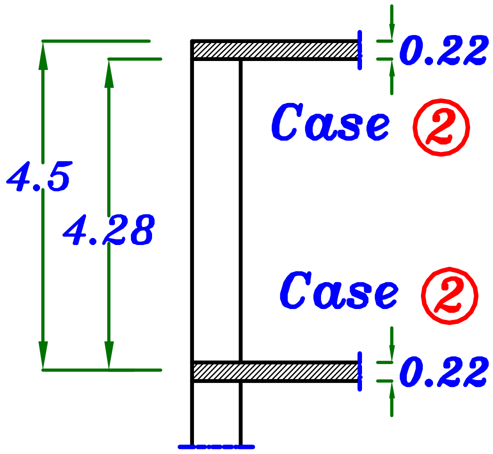
$$\boxed{M_{ext} = M_C = 67.59 \ \ kN.m}$$ يؤخذ العزم كما هو لانه أخر دور



يجب أن لا تقل b عن $500\ mm$
حتى تكون البلاطه Safe Punching

Take $\boxed{b = 500\ mm}$

Check Buckling.



العمود متماثل فى الاتجاهين

Upper Case ②
Lower Case ②
$\Big\} \ k = 1.5$

$$H_o = 4.28\ m$$

$$\lambda_b = \frac{1.5 * 4.28}{0.50} = 12.84 > 10$$

$$\delta = \frac{(\lambda_b)^2 * b}{2000} = \frac{12.84^2 * 0.5}{2000} = 0.041\ m$$

$$M_{add.} = P * \delta = 423.06 * 0.041 = 17.34\ kN.m$$

$$M_{des.} = M_{ext.} + M_{add.} = 67.59 + 17.34 = 84.93\ kN.m$$

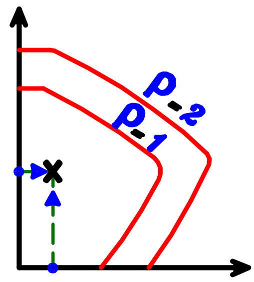
$$e = \frac{M}{P} = \frac{84.93}{423.06} = 0.20\ m \qquad \therefore \frac{e}{t} = \frac{0.20}{0.50} = 0.40 \xrightarrow{use} I.D.$$

$$\zeta = \frac{500 - 100}{500} = 0.80 \xrightarrow{use}$$ ECCS Design Aids Page 4–24

$$\frac{P_U}{F_{cu}\, b\, t} = \frac{423.06 * 10^3}{30 * 500 * 500} = 0.056$$

$$\frac{M_U}{F_{cu}\, b\, t^2} = \frac{84.93 * 10^6}{30 * 500 * 500^2} = 0.022$$

$\Big\} \ \rho = 1.0$



$$\mu = \rho * F_{cu} * 10^{-4} = 1.0 * 30 * 10^{-4} = 3.0 * 10^{-3}$$

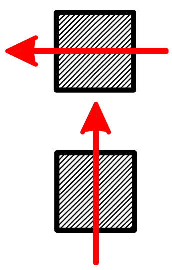
$$A_s = A_{s`} = \mu * b * t = 3.0 * 10^{-3} * 500 * 500 = 750 \ mm^2$$

$$A_{s_{Total}} = A_s + A_{s`} = 2 * 750 = 1500 \ mm^2$$

$$A_{s_{min}} = \frac{0.25 + 0.052 \lambda_{max}}{100} * b * t$$

$$= \frac{0.25 + 0.052 \ (12.84)}{100} * 500 * 500 = 2294.2 \ mm^2 > A_{s_{total}}$$

$$Take \quad A_s = A_{s`} = \frac{A_{s_{min}}}{2} = 1147.1 \ mm^2 \qquad \boxed{6 \phi 16}$$



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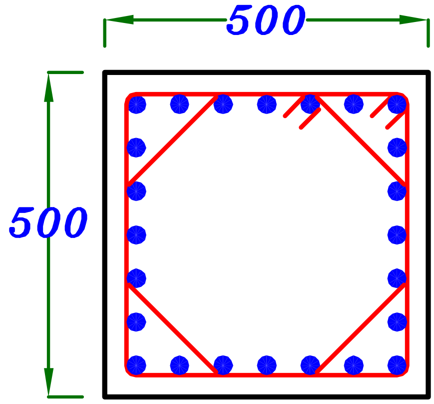
و لم يتم عمل حالات تحميل للاتجاه الاخر .



$24 \phi 16$

عدد الاسياخ ٦ اسياخ فى كل جنب
المجموع الكلى للاسياخ ٢٤ سيخ
نضع ٤ اسياخ فى الاركان و الباقى يوزع على الاربع جناب .